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Evaluation of Static Low Density Media filters for use in domestic wastewater treatment

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EVALUATION OF STATIC LOW DENSITY MEDIA FILTERS FOR USE IN DOMESTIC WASTEWATER TREATMENT

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"*The development of wastewater treatment has to a very large extent been hampered by tradition*."

– Hallvard Ødegaard, 2000

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ABSTRACT

Static Low Density Media (SLDM) filters are submerged granular medium filters that contain a static matrix of floating media. These filters provide concurrent biological and physical treatment, and are therefore classified as bioclarifiers. Through different design and operation strategies, SLDM filters may be used for a variety of functions such as: solid-liquid separation alone, organic conversion and solids capture, nitrification and solids capture, and denitrification and solids capture. For operation as an aerobic unit, an external aeration strategy was developed to preserve the static nature of the bed.

In this study, SLDM filters treated a highly variable flow domestic wastewater generated from an industrial facility in Denham Springs, Louisiana. Various bench scale filter configurations were evaluated on their ability to perform both biological and physical treatment at a variety of hydraulic filtration rates, backwash frequencies, and configurations, while constantly keeping the filter bed in an aerobic state. Data collected from units recirculated via airlift pumps is reported. The pneumatically washed units in this study employed a modified shape media and a high backwashing frequency to enhance biofiltration capacities. Units were fed primary domestic wastewater effluents with mean $CBOD_5$ (carbonaceous biochemical oxygen demand) of 100 and 150 mg/L. Mean influent TSS (total suspended solids) values for the two units tested were 60 and 90 mg/L. The airlift/SLDM filter combination was able to maintain mean hydraulic filtration rates in the range of 10-15 m/hr.

Findings indicate the unit is capable of producing $CBOD_5$ and TSS effluent qualities in the 10-20 mg/l range when subject to organic loadings between 1 and 3.5 kg/m^3.day. These values are higher than reported loading capacities for conventional secondary wastewater treatment strategies, such as Activated Sludge units, Trickling Filters, and Biological Aerated

Filters. In this study, effluent $CBOD_5$ levels were closely correlated with effluent TSS levels. Although no problems with media caking were observed, at times poor backwash interval selection did lead to severe oxygen depression within the bead bed. It is concluded that SLDM show promise for application in the domestic wastewater arena, particularly where the scale of the operation places a premium on simple operation.

CHAPTER 1 : INTRODUCTION

The engineered treatment of wastewater stretches only slightly beyond one hundred years. Prior to this, wastewater was simply disposed of and treatment occurred naturally. It was not until London's Broad Street Well cholera epidemic of 1854 that the link between contaminated water and communicable disease was made (Babbitt and Doland, 1931). In a world that had not yet witnessed the discoveries of Louis Pasteur, this revolutionized the treatment of drinking water. A few decades later, the treatment of wastewater prior to disposal emerged as an additional means to protect human health. The focus of wastewater treatment extended from physical treatment to engineered aerobic systems for biological treatment, a relatively recently development. In 1925, only twenty percent of United States cities with populations greater than 100,000 had wastewater treatment facilities (Linsley and Franzini, 1972). Today, practically all cities have wastewater treatment of some form; in cities greater than 100,000, wastewater treatment is typically centralized in large plants.

Large centralized treatment plants employ sewer collection systems to transport the majority of the wastewater generated in the city to a single location, where it is treated and subsequently discharged. Centralized systems usually consist of several specialized treatment components in series to treat the wastewater, which can consist of both household and commercial wastewater. The core of the classical unit operation and process configuration consists of primary clarification, biological treatment, and secondary clarification, separated into individual physical, biological, and chemical units. Numerous variations in the configurations and technologies used in centralized systems have resulted in a wide variety of treatment mechanisms possible, although the overall philosophy of solids removal followed by dissolved organic material and then biomass removal is consistent.

The unit operation and process strategy is recurrent throughout all forms of wastewater treatment, whether it occurs in a publicly owned treatment works (POTW) facility that services a population equivalent of three hundred thousand, a smaller package plant for a neighborhood community, or an onsite system for an individual household. When connection to a municipal line is not possible (e.g. remote areas), conventional onsite systems are typically used. The most widely used apparatus for the decentralized treatment strategy, the practice of treating and then discharging wastewater within the local vicinity of where it was generated, consists of a septic tank followed by a soil absorption field (Jowett and McMaster, 1995). Such systems are used to sequentially remove solids and then organic material, and are known to work well if they are properly designed, located, installed and maintained. Suitable conditions are often not met, resulting in incomplete treatment. Two-thirds of all the land area in the United States is estimated to be unsuitable for the installation of septic tanks and their drain fields (USEPA, 2002; Linsley and Franzini, 1972).

Effective wastewater treatment is essential to protect both human and environmental health, regardless of the size of the community. Potential contaminants in domestic wastewater include disease-causing bacteria, infectious viruses, household chemicals and excess nutrients such as ammonia, along with the more traditional suspended solids and biochemical oxygen demand. Both centralized treatment plants and onsite systems must consistently yield effluents that have minimal quantities of such contaminants in order to reduce local impact. Failure of onsite systems, particularly with regard to hydraulic overloading of subsurface drain fields and the detrimental effects of this failure on receiving waters, has been documented in the past (USEPA, 2002; Chaffe, 2000; Hagedorn et al., 1981). Wastewater treatment effluent can be drawn into groundwater that may be used for recreational purposes or as a drinking water source, resulting in a potential hazard. The potential for disease transmittal by this means is real. In the United States between January 1991 and December 2000, 142 epidemiological outbreaks of disease due to contaminated drinking water were confirmed and reported by the Center for Disease Control, along with 193 outbreaks due to contaminated recreational waters (Lee *et al*., 2002; Barwick *et al*., 2000; Levy *et al*., 1998; Kramer *et al*., 1996; Moore *et al*., 1993). This resulted in nearly 450,000 reported illnesses, including the largest outbreak of cryptosporidiosis in United States history, which affected over 400,000 people and caused 4,000 hospitalizations (USEPA, 1997). Environmental impacts, such as eutrophication and algal blooms, have also resulted due to discharges into surface waters from sources such as centralized systems (Münch *et al*., 2000).

As worldwide population increases and cities experience rapid growth, wastewater treatment needs will increase, and pre-existing plants will require expansion and further development. Without careful planning or adequate design, increased urbanization could have adverse impacts on environmental quality, resulting in the loss of valuable natural resources and diminished overall aesthetic appeal. An increase in population density has and will exert further demands on the wastewater treatment industry to provide technologies capable of reducing environmental impact while increasing economic efficiency. Factors critical to the selection of operations and processes for treatment plants upgrading to meet the needs of growing communities include: area requirements, cost, treatment efficiency, and sustainability (Ødegaard, 2000). The culmination of these demands on the wastewater treatment industry has resulted in a need for innovative design of new units and creative ideas for modifying older, preexisting plants. As large wastewater treatment plants attempt to expand or upgrade systems, mono-functional technologies currently in use, such as the activated sludge process, have proven difficult to update (Bigot *et al*., 1999). While the unit operation and process approach to wastewater treatment is effective, it results in systems requiring a large footprint, high capital resources, and sophisticated staffs.

Consolidation of biological processes and physical operations into a single unit represents the ability of a single technology to function as an entire secondary treatment train, provided appropriate treatment levels are obtained. Depending on the application, further possibilities exist, such as the potential of a single unit to act as a primary and secondary treatment system, or as a secondary and tertiary unit. Pressure to develop infrastructure with limited space and resources has resulted in the footprint of a wastewater treatment plant becoming a more important parameter in the construction of systems (Ødegaard and Helness, 1999). Consolidated units would fulfill requirements of a small footprint in today's land-conscious society. While individual processes efficiencies may decrease in consolidated units, the savings in capital and operating costs for single components serving multiple duties may overcome the disadvantages (Loyless and Malone, 1998). Evaluation of such a unit could determine the extent of efficiency loss and would lead to potential uses and the position of the unit in the spectra of wastewater treatment systems. A treatment system designed within the consolidation strategy could provide both large and small communities an attractive alternative to current systems.

Recent developments in fixed film technologies have focused on consolidating the treatment train through utilization of technologies that can perform more than one operational process. Submerged biofilters have been suggested as a small footprint alternative for new construction and renovation of existing plants, noting further advantages of stability and multiple functionalities (Sampa and Tanaka, 1995). A promising apparatus for this application is the Static Low Density Media (SLDM) filter. SLDM filters, such as the one shown in Figure 1-1, are a class of submerged granular medium filters that contain support media for the development of biofilm and provide external aeration as well as recirculation for aerobic microbial processes. The granular biofilm carriers used in SLDM filters have a specific gravity of less than one, which results in a floating bed of filter media. The media bed is operated in a static mode to promote solid-liquid separation in addition to biofiltration. Concurrent physical and biological treatment, as performed in this robust, yet eloquently simple filter, follows the ideals of the consolidation strategy.

Figure 1-1. A commercially available Static Low Density Media (SLDM) Filter (AST) currently used in the aquaculture industry may have applications for wastewater treatment. The granular media used for bioclarification floats at the top of this submerged filter.

It was the overall goal of this research to determine if the SLDM filter could be successfully used for secondary treatment of domestic wastewater. In the document that follows, specific objectives, results, discussion, and conclusions are delineated in a series of chapters. Chapter 2, entitled ‘Background’ describes the SLDM filter as one of several options for biofiltration, as well as a means of solid-liquid separation. Specific objectives of this study are also presented in Chapter 2. Chapter 3, entitled ‘Static Low-Density Media Filter for Removal of Solids and Organics from a Variable Flow Domestic Wastewater Source’ describes the operation and performance of the prototypes used in this study. Chapter 4, ‘Management Strategies for Static Low Density Media Filters Treating Domestic Wastewater’, focuses on the impact of various operational parameters that are controllable in SLDM filters. Chapter 5, entitled ‘Global Discussion and Conclusions’, presents the conclusions drawn from this study and suggests possible future research plans.

CHAPTER 2 : BACKGROUND

The traditional approach taken to domestic wastewater treatment has been the serial removal of solids, followed by dissolved organic material, and then biomass. Individual units comprise the conventional treatment train used to treat domestic wastewater, and they are divided into a serial sequence of pretreatment, primary treatment, and secondary treatment, followed by disinfection. Pretreatment involves screening the wastewater, typically with bar racks, and grit removal. Bar racks are used to remove large materials that are inadvertently present in the wastewater, such as pieces of plastic, sticks, or rags, which may clog or damage subsequent units, while grit chambers remove dense inert material that may cause abrasion to units downstream in the treatment train. Following pretreatment is primary treatment, a solid-liquid separation step. Next is secondary treatment, the biological conversion of dissolved organic compounds to bacteria and the subsequent removal of the generated biomass. Finally the wastewater is disinfected and discharged to a receiving body of water.

The unit operation approach has induced a divided focus to wastewater treatment, as units and processes are optimized to treat individual target parameters. Conventionally, physical and biological treatment steps have been separated and placed in series. The resulting systems, while technically sound, require a large footprint and considerable capital investments from the communities that depend on them. Combining physical and biological treatment capacity into single units may simplify complex treatment trains while providing additional economic and footprint advantages. This consolidation strategy has been used in the development of the Static Low Density Media (SLDM) filter, in which biological and physical treatment occurs concurrently.

Clarification of Wastewater

Particle separation is a major component of domestic wastewater treatment. Most of the pollutants in wastewater exist in suspended or colloidal form, or are transformed to this form during the course of the treatment process (Ødegaard, 1998). Physical treatment operations are employed multiple times throughout the traditional domestic wastewater treatment train, primarily to reduce the concentration of these solids in wastewater, although any means of altering the physical characteristics of water, such as gas transfer or flow metering, may constitute physical treatment.

Solids separation is performed in most wastewater treatment plants by sedimentation of wastewater in large clarifiers, frequently referred to as settling tanks, settling basins, or sedimentation basins. The primary driving force in sedimentation is the difference in specific gravity between the liquid and solid phases. Particles settle out of suspension as the wastewater passes through the primary sedimentation basins, typically at an average overflow rate of 2 to 3 $m^3/m^2.h$, and with an average detention time of two hours (Liao and Ødegaard, 2002; Metcalf and Eddy, 1991). During this time, approximately 50 percent of the total suspended solids (TSS) are removed (Ødegaard, 1998). Primary treatment is critical to minimizing the load on subsequent biological treatment steps, as a portion of the solids removed in this step represent particulate bound BOD (PBOD). Measurements on several wastewater influents have shown that up to 70 percent of influent COD is related to particles greater than 0.45 microns in diameter (PCOD), and many contaminants are incorporated into or adsorbed onto particulate material (van Nieuwenhuijzen *et al*., 2001). In addition to TSS removal, settling basins in primary treatment are responsible for removing 25 to 40 percent of the BOD in typical domestic wastewater (Metcalf and Eddy, 1991).

Clarification is also required after most biological treatment units to remove the biological floc created during the removal of soluble organic material. Most large treatment plants use sedimentation for this purpose, although the large footprint requirement of secondary sedimentation basins is prompting designers of these systems to engineer an alternate means of solid-liquid separation.

FILTRATION

Filtration is considered the most important solid-liquid separation process in water treatment, as well as in tertiary wastewater treatment (Zouboulis *et al*., 2002; Ngo and Vigneswaran, 1995). It is currently being evaluated for additional applications to domestic wastewater treatment and is gaining attention as an alternate technology for secondary clarification of wastewater. Filtration allows an increased level of design control to the engineer as compared to sedimentation (Svarovsky, 1977). While several classes of filters have been developed, granular medium filters have emerged as showing excellent potential for wastewater treatment (Ødegaard and Helness, 1999).

Granular medium filters separate solids from liquids by passing the suspension through a permeable granular medium, which retains the particles. Mechanisms within granular medium filters that contribute to the removal of suspended solids include: straining, sedimentation, impaction, interception, adhesion, chemical adsorption, physical adsorption, flocculation, and biological growth (Metcalf and Eddy, 1991). These filters can be arranged in various configurations with several different options for media type. The filters may operate in packed or expanded modes with flow direction either up or down flow. The medium may be sand, anthracite, gravel, clay, or a variety of plastics including polyethylene, styrofoam, and biodegradable polyhydroxyalkanoates (PHAs). Flow may be either pressurized or by gravity.

Granular packed bed filters have shown a natural propensity for biological treatment in addition to clarification. Biofilms tend to attach to granular media and it is not probable that they will be completely removed during backwashing in filters designed solely for solid-liquid separation. Tendency to behave like a fixed film bioreactor often causes difficulties in sand filters faced with high organic loadings. Reductions of both oxygen concentration and soluble COD concentration in other types of granular medium filters have further supported this claim (Ødegaard, 1998).

Sand Filters

Sand filters are granular medium filters, which may be packed or fluidized, downflow or upflow, single pass or recirculating, that contain sand as the filtering medium. Packed sand filters have been widely used for their filtration abilities; they remove particulate matter via the mechanisms of physical straining, adsorption, and biodegradation (Jellison *et al*., 2000). While sand filters have been used for their filtration capabilities to yield potable water for over 150 years, they have only recently been applied for domestic wastewater treatment (Sadiq *et al*., 2003). Sand filter applications in wastewater treatment are most commonly for the removal of biological floc from secondary treatment effluent prior to discharge into receiving bodies (Metcalf and Eddy, 1991). Sand filters have also been used to reduce the total numbers of fecal coliforms in the effluent from secondary wastewater treatment units prior to reuse of the wastewater in agriculture applications (Sadiq et al., 2003).

Floating Granular Medium Filtration

For the past fifteen years, there has been interest in using floating granular media filtration for high rate particle separation, particularly in Northern Europe and Japan. There has been repeated demonstration of success of floating media filters in the removal of suspended solids, both with and without pre-coagulation, from both raw domestic wastewater (Liao and Ødegaard, 2002; Sampa and Tanaka, 1995; Tanaka *et al.*, 1995; Mouri and Niwa, 1993) and effluent from secondary treatment (Ødegaard and Helness, 1999; Ngo and Vigneswaran, 1995).

Configuration of low-density media filters is typically with the water flowing up through the granular filtration bed. The medium used in these filters is typically coarse (2–10 mm), therefore having high porosities and high sludge accumulation capacities (Ødegaard and Helness, 1999). In fact, it has been claimed that floating medium filters have higher retention capacities and lower headloss development when compared to conventional sand filtration (Ngo and Vigneswaran, 1995). Operation at high filtration rates of 5 – 50 m/h, lengthy filter run times, low head-loss, and ease in backwashing are some of the additional advantages low-density media have to offer. Efficiencies of 70 to 85 percent removal of suspended solids have been reported in floating KMT media filters used solely for solid-liquid separation (Ødegaard and Helness, 1999). Removal efficiencies from a floating media filter employing a different media type and pre-coagulation are presented in Table 2-1.

Table 2-1. Higher solids captures were observed at lower filtration rates in granular floating medium filters used to treat domestic wastewater effluent from a secondary treatment unit. The media had a diameter of 3.8 mm and a porosity of 0.36. (Ngo and Vigneswaran, 1995).

	Average Removal (%)	
Parameter	5.4 m^3/m^2.hr	2.5 m^3/m^2.hr
Turbidity	55	86
Suspended Solids	35	59
Color (apparent)	41	69
Orthophosphate	61	74

Biofiltration of Wastewater

Biological treatment, or biofiltration, processes utilize the metabolic capacity of viable microorganisms in order to convert substrates to chemically and/or physically different products.

In domestic wastewater treatment, these processes are used to reduce the dissolved organic matter, or soluble biochemical oxygen demand (SBOD), of the wastewater. The predominate means used is the conversion of SBOD to particulate bound BOD, which may be removed by subsequent clarification techniques. Biological processes are also used for nitrification and denitrification, although fast growing aerobic, chemoheterotrophic bacteria tend to dominate when organic levels are high.

Over the past century the mainstays of domestic wastewater biological treatment in centralized plants have been the activated sludge process and the trickling filter (Stephenson *et al.,* 1993). These two processes represent the archetype of suspended growth and fixed film systems, respectively. Other biological processes exist within these two categories, although none have yet achieved comparable levels of acceptance or use in large centralized wastewater treatment plants. However, within the past two decades there has been considerable development and recognition of high rate biological fixed film processes.

Suspended Growth

Suspended growth systems stir and suspend microorganisms in wastewater. As the microorganisms absorb organic matter and nutrients, they grow in size and number, resulting in flocs of biomass. The bacterial culture is maintained in suspension in an aerobic environment. After a specified retention time, typically on the order of several hours, bacteria are passed to a clarifier where the microorganisms are settled out as sludge, some of which is pumped back into the incoming wastewater to increase the amount of biosolids held in the reactor. The remaining sludge is wasted and sent on to a sludge treatment process. Suspended growth processes used to reduce carbonaceous BOD (CBOD) include the following: the activated sludge process, aerated lagoons, sequencing batch reactors, and the aerobic digestion process.

FIXED FILM

Fixed film systems, also known as attached growth systems, are biological treatment processes that use supporting medium as a physical substrate for biomass growth. Biofilm, a community of microorganisms embedded in a matrix of extracellular biopolymers, develops on a supporting substrate (also known as the media or as a biofilm carrier) and remains fixed to it as wastewater is passed over the medium. The biofilm absorbs organic matter and nutrients from the passing wastewater. Examples of fixed film systems include: trickling filters, rotating biological contactors, submerged media filters, and biofilters with packed or fluidized granular medium beds, such as fluidized sand filters.

Fixed film processes have several favorable characteristics that make them attractive candidates for use in municipal wastewater treatment. Biofilms have the potential to have a greater treatment efficiency on low concentration wastewaters than activated sludge units, and can therefore be used for achieving very stringent effluent guidelines and regulations (Iwai and Kitao, 1994; Rittmann and Brunner, 1984). The microbial consortia in biofilm systems are more diverse than those of conventional activated sludge systems (Bishop, 1997). Since more biological niches exist in a biofilm system, organisms that would not otherwise be able to survive or compete in a completely mixed suspended growth system are able to become established and thrive (Bishop, 1997). Operation and maintenance of fixed film biofilters can result in a preferential growth of microbial films, allowing specialized treatment of wastes (Boller *et al.*, 1997; Ohashi *et al.,* 1995; Ødegaard *et al.*, 1994).

As opposed to activated sludge units, fixed film biofilters typically require a smaller footprint and are less dependent on final sludge separation (Rodgers, 1999; Sampa and Tanaka, 1995; Boller *et al.*, 1994; Ødegaard *et al.*, 1994). Fixed film systems are not only generally less energy-intensive and more resistant to shock loadings and other changes in process parameters, but they also have a higher removal rate of organic pollutants per unit volume of apparatus as compared to suspended growth systems (Hu *et al*., 2001; Chaudhry and Beg, 1998).

Granular medium biofilters contain the advantages of fixed-film processes, along with the favorable characteristics of granular medium. Granular medium, in contrast to a fixed or corrugated medium, may be operated in packed, expanded, or fluidized modes. Granular medium typically contains higher specific surface areas and lower porosities, thereby decreasing the volume and footprint requirements of the bioreactor. The net volumes of granular media biofilters are more than ten times smaller than those of activated sludge systems (Boller *et* al., 1994). In comparison to conventional biofilm systems such as trickling filters or rotating biological contactors, granular medium biofilters may require two to three times smaller construction volumes (Boller *et al*., 1994).

SUBMERGED AERATED FILTERS

Submerged Aerated Filters (SAFs), also known as Submerged Contact Aerators, are typically used for biofiltration of settled domestic wastewater. SAFs are fixed-film reactors, which utilize a high-voidage media and air scour to prevent solids accumulation and thus the need for backwashing (Hodkinson *et al*., 1999). Compressed air is introduced through perforated pipes beneath the contact media, which may be granular or fixed blocks (Rusten, 1984). The strong turbulence generated by the vigorous aeration abrades the biofilm on the medium and distributes the substrate evenly throughout the filter. Residence time distribution curves from tracer studies indicate that SAFs may be considered completely mixed reactors (Rusten, 1984). Since influent solids and generated biomass are not trapped by the filter medium, SAFs must be followed by a clarifier.

SAFs are currently being used in package wastewater treatment plants. The typical organic loading rate applied to SAF bioreactors is 0.43 kg BOD/m^3.d (Hodkinson *et al*., 1999). Some proprietary SAFs and have been listed in Table 2-2.

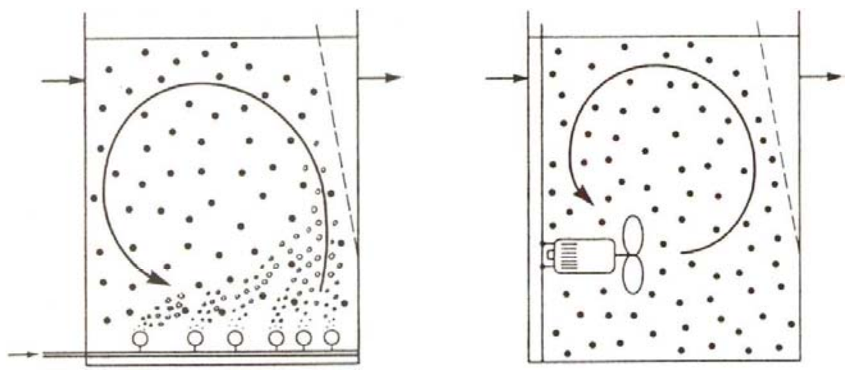
Table 2-2. Examples of SAF processes on the market (after Stephenson *et al*., 1993).

Process	Manufacturer	Influent Flow	Support Media	Media Type
BAF	Copa	Down/up	Plastic	Structure
CTX	Hodge Stetfield	Down/up	PVC	Structure
Fast	Promech	Downflow	Polyurethane	Structure

While submerged aerated filters fall under the technical definition of Biological Aerated Filters (BAFs) with the exception of backwashing, they have been segregated from the general classification of BAFs as result of their inability to perform concurrent biofiltration and solids capture. Such filters will be discussed in a later section. Moving Bed Biofilm Reactors (MBBRs), which fit the loose characterizations of both SAFs and BAFs, have been presented separately in the following section, due to both the specificity of the MBBR definition and their ability to function as both aerobic and anaerobic units.

MOVING BED BIOFILM REACTORS

Moving Bed Biofilm Reactors (MBBRs) are continuously operating fixed film granular medium biofilters, which employ biofilm carriers with specific gravities of less than one and high specific surface areas, typically granular Kaldnes (KMT) media. The MBBR is one of the latest developments in a class of reactors situated in the spectra of wastewater treatment between activated sludge and fixed-film biofilters (Maurer *et al*., 2001). Biofilm is attached to the carrier elements that move freely along with the water in the reactor, as shown in Figure 2-1 (Rusten *et al*., 1997; Ødegaard *et al*., 1994). The process has been tested in pilot and full scale plants since 1989 for treatment of municipal wastewater and wastewater from the food industry (Broch-Due *et al*., 1994), and today there are over 100 plants around the world that use MBBRs for various treatment purposes such as organics removal, nitrification, and denitrification in both municipal and industrial wastewater (Ødegaard *et al*., 2000). MBBRs have also been favorably used to upgrade activated sludge plants, and have shown a higher rate of soluble organic removal per volume and a higher rate of nitrification than traditional activated sludge plants (Münch *et al*., 2000; Ødegaard *et al*., 1994). Unlike the activated sludge process, sludge is not recycled in MBBRs.



a. Aerobic reactor b. Anoxic and anaerobic reactor

Figure 2-1. Moving Bed Biofilm Reactors are filled with a low-density media that moves freely with the water in the reactor (from Ødegaard, 2000).

MBBRs may be aerobic or anaerobic/anoxic reactors. In the former case, the biofilm carriers are moved by aeration, whereas in the latter case, mechanical mixing is used to agitate the biofilm carriers. As the beds of MBBRs are fully dynamic, there is no need for backwashing; the headloss through the reactor is insignificant as result of the turbulent mixing (Maurer *et al.*, 2001; Ødegaard *et al*., 1994). As such, a secondary clarifier is necessary after a MBBR. Studies have reported poor solids capture of 30 to 55% (Broch-Due *et al*., 1994). Consequently, the MBBR may only be used for biofiltration. Typical operating parameters for Moving Bed Biofilm Reactors are listed in Table 2-3.

Table 2-3. Typical operating parameters for Moving Bed Biofilm Reactors (MBBRs).

Variable	Range	Source
Media		
Media Diameter (mm)	7 – 15	(Andreottola et al., 2000;
Media Length (mm)	10 – 15	Ødegaard et al., 2000;
Specific Surface Area (m^2/m^3)	160, 490 – 7700	Ødegaard, 2000)
Filling Fraction (%)	60 – 70	
Contact Time (hr)	2 – 5	(Andreottola et al., 2000; Ødegaard et al., 2000)
BOD Loading Rate (kg $BOD_7/m^3.d$)	4 – 5	(Ødegaard, 2000)
Backwashing	Never	
Maximum Headloss	Insignificant	

Bioclarifiers

A bioclarifier is an apparatus that not only uses a physical medium as a biofilm carrier for fixed film growth, but also simultaneously uses the filtration capacity of the media for its solid-liquid separation abilities. Simplicity, efficiency, and flexibility are characteristics that should be incorporated into successful bioclarifier design. Bioclarifiers currently in use in domestic wastewater treatment include: Biological Aerated Filters (BAFs) and some recirculating sand filters.

The media used in bioclarifiers are typically granular in nature. In general, granular media capture suspended solids not only more effectively, but also in greater amounts than other shaped media (Iwai and Kitao, 1994). Furthermore, plastic medium has been increasingly used in granular medium bioclarifiers, as plastic extrusion processes allow a greater control over size, shape, and density of biofilm carriers (Mann *et al*., 1995). Both suspended solids removal and dissolved organics removal are affected by these media properties. Shape, size, and density influence the effective and available biofilm surface area for growth, solids capture efficiency, backwashing requirements and frequency, quantity of producible product water, and hydrodynamics within the bioclarifier.

The use of floating media in upflow filters has eliminated the possibility of fluidization of media and has increased the range of applicable filtration rates. Additional characteristics of floating medium include low head loss during filtration and less energy required for backwashing as compared to sunken medium (Tanaka *et al.*, 1995). In studies comparing sunken and floating media, it was determined that wastewaters with high levels of suspended solids would be best treated using floating medium upflow reactors, and that floating medium outperformed sunken medium for COD and ammonia removal in addition to TSS removal (Mann *et al.*, 1998). The main mechanisms that contribute to the removal of suspended matter in a floating filter are: straining, interception, flocculation, sedimentation, and adhesion to biofilm growth (Ødegaard and Helness, 1999).

Along with biological conversion, biofilms are also known to capture particles, thereby potentially providing an additional level of physical treatment, although this natural ability must be actively incorporated into bioclarifier design in order to derive any benefit. Iwai and Kitao (1994) described particle capture in biofilm itself as occurring through: interception, inertia, gravitation, electrical charge, diffusion, and hydrodynamic means. In bioclarifiers, organic particles may be captured in interstitial spaces of the media. These particles may be flushed out of the filter by a backwashing mechanism; however, they may undergo hydrolysis prior to sludge discharge, which would provide the microbial community with an additional source of soluble organic matter.

Aerobic bioclarifiers must incorporate both sufficient aeration and residence time in their designs to improve biofiltration performance. This may be achieved by providing either internal or external aeration. If external aeration is used, multiple-passes, or recirculation, through the unit must be made. Recirculation minimizes impact on the clarification capacity of the media, while introducing benefits of reduced size and increased stability. Recirculating filters are smaller and more robust than single-pass filters and can exert greater performance control, as recirculation ratios can be altered to optimize treatment (USEPA, 2002). The basic components of a recirculating bioclarifiers include the recirculation tank, pump, and the bioclarifier.

ON-SITE SAND FILTERS

In smaller scale, on-site wastewater treatment installations, sand filters have been used as an auxiliary unit for additional treatment of wastewater. Auxiliary units for conventional onsite systems have been developed in an effort to reduce failure rates and combat the detrimental effects of failed septic systems. These units are bioclarifiers; they reduce both CBOD and TSS, along with viable bacteria in the effluent applied to the soil, thereby allowing higher soil loading rates and less dependence on the soil drain field for treatment. Auxiliary units not only reduce failure rates of conventional systems, but could also reduce the introduction of enteric pathogens to groundwater (Pundsack *et al*., 2001).

Various studies have shown that single stage, intermittent sand filters with BOD_5 loadings of 10 $g/m^2.d$ and hydraulic loadings of 5 – 10 cm/d are capable of removing 95% of the BOD_5, 30% of the total nitrogen, and 99 – 99.9% of the fecal coliforms from household domestic wastewater (van Buuren *et al*., 1999). These studies have also shown that filter efficiency could be increased by recirculation of filter effluent and by amending the sand with absorbents or by replacing sand with plastic media (van Buuren *et al*., 1999). Chaffee (2000) noted the most common technologies for auxiliary unit applications are aerobic treatment units, attached-growth media filters, and natural systems, and suggested use of a recirculating media biofilter, such as a recirculating sand filter.

The USEPA (2002) has stated that recirculating packed bed filters, in particular sand filters, are extremely effective and reliable in removing both BOD and TSS from domestic wastewater. Recirculating bioclarifiers have been found to be a cost effective solution for failed septic systems in many parts of the United States (Chaffee, 2000). Recirculating fixed film filters have also been shown to provide advanced secondary treatment of wastewater, and may provide nitrification under favorable operating conditions.

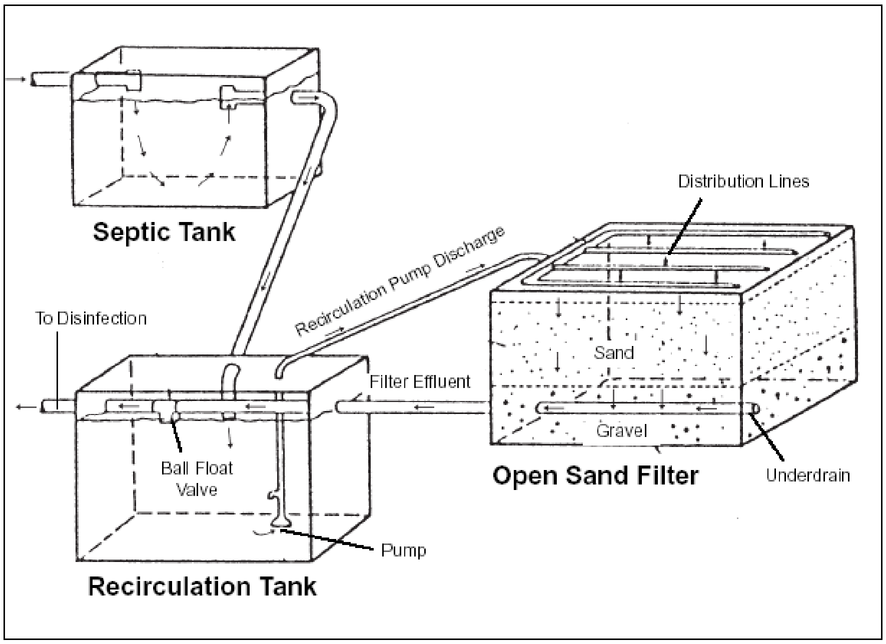


Figure 2-2. A typical recirculating sand filter system for on-site treatment consists of septic tank, recirculation tank, and sand filter components (USEPA, 2002).

Recirculating sand filters have been applied as a means to pretreat wastewater prior to subsurface infiltration at locations such as single-family residences and commercial establishments where soil conditions are unsuitable for direct discharge (USEPA, 2002). Recirculating sand filters have also been successfully used in conjunction with residential septic systems to reduce the nitrogen loading of receiving bodies (Piluk and Byers, 2001). Typical design specifications for individual home recirculating sand filters may be found in Table 2-4.

Table 2-4. Typical design specifications for on-site recirculating sand filters (USEPA, 2002).

Design Parameter	Typical Design Value
Media	
Sand Media Diameter (mm)	1.0 – 5.0
Gravel Media Diameter (mm)	3.0 – 20.0
Bed	
Depth (cm)	46 - 91
Hydraulic Loading ($m^3/m^2.d$)	
Sand	0.12 – 0.20
Gravel	0.41 – 0.61
Organic Loading ($g/m^2.d$)	
Sand	≤ 22.8
Gravel	≤ 68.3
Recirculation Tank	
Volume	1.5 times design of daily flow
Recirculation Rate	3 to 5 times daily flow

BIOLOGICAL AERATED FILTERS

The term Biological Aerated Filter covers a broad range of designs and configurations (Mann *et al*., 1995). A BAF is generally defined as a submerged fixed film filter in which air or oxygen is introduced below or inside of the filter bed and passes upward, through the media support. The media used within a BAF provides sites for biological growth and removal of biomass (Stephenson *et a*l., 1993). Some definitions of "true" BAFs require the media be granular in nature with a large specific surface area (Moore *et al*., 2001). Nevertheless, internal aeration, the common BAF theme, causes the beds of BAFs to be pulsed or semi-static. BAF beds typically function as bioclarifiers, concurrently removing dissolved organics and solids (Metcalf and Eddy, 2003; Chen *et al.,* 2000; Pujol *et al*., 1994; Stephenson *et al*., 1993). BAFs have been used for a variety of applications including: simultaneous removal of both solids and BOD, combined BOD removal and nitrification for secondary upgrading, and tertiary treatment for combined BOD removal, nitrification, and simultaneous solids filtration.

Originally developed for drinking water filters, there are currently hundreds of full-scale plants that treat a variety of different wastes including domestic wastewater, industrial wastewater, agricultural wastewater, and drinking water. The first full scale plants were built in the 1980s, and can now be found mainly in the United States, Canada, Japan, and several European countries (Osorio and Hontoria, 2001). Many different types of BAFs have been developed, operating in both an upflow or downflow mode, using various types and sizes of media. BAF media are typically granular in nature and have traditionally consisted of natural materials, although synthetic materials have been more recently used (Kent *et al.*, 1996). Examples include: polyurethane foam cubes (Chandravathanam and Murthy, 1999), low density granular polypropylene (Yoo and Kim, 2001; Mann *et al.*, 1998), floating polyethylene (Belgiorno *et al.*, 2003), low density granular polystyrene (Payraudeau *et al.*, 2000; Le Tallec *et al.*, 1997), ceramic-based material (Osorio and Hontoria, 2001), and vitrified clay (Stensel *et al.*, 1988). Other design differences, such as co-current backwashing for denser medias versus counter-current backwashing for floating medias, exist. While there are a large variety of different proprietary designs of BAFs on the market, little difference in terms of performance among the BAFs has been observed (Stephenson *et al.*, 1993). Various examples of proprietary BAF processes may be found in Table 2-5.

Table 2-5. Examples of backwashing BAF processes on the market (after Hodkinson *et al.*, 1999).

Process	Manufacturer	Influent Flow	Support Media	Media Type
Biobead	Brightwater	Upflow	Polyethylene	Floating
Biocarbone	OTV/Degremont	Downflow	Expanded Shale	Sunken
Biofor	Degremont	Upflow	Biolite	Sunken
Biopur	Sulzer/John Brown	Downflow	Polystyrene	Modular
Biostyr	OTV/GWP	Upflow	Polystyrene	Floating
ColOX	Tetra	Upflow	Sand	Sunken
SAFe	PWT Projects	Downflow	Expanded Shale	Sunken
Stereau	Purac	Downflow	Pumice	Sunken

Headloss due to clogging has been identified as one of the most important parameters for the operation and design of BAF systems (Le Tallec *et al.,* 1999). Backwashing is the means by which headloss is reduced in a BAF; therefore, the systems operate in a series of filtration and backwash cycles. Backwashing in BAFs is typically hydraulic, usually requiring ten percent of the product water to be used for this purpose (Stensel *et al*., 1988). In fact, it has been suggested that a backwash flush water equalization tank should be a necessary element in a treatment plant with a BAF system, since backwash water could result in a significant hydraulic surge to primary clarifiers (Stensel *et al*., 1988).

Table 2-6. Typical Characteristics of Biological Aerated Filters.

Variable	Range	Source
Media		
Media Diameter (mm)	3 – 6	(M'Coy, 1997)
Specific Surface Area (m^2/m^3)	500 – 2000	(M'Coy, 1997)
Loss (amount/year)	< 2%	(M'Coy, 1997)
Bed Depth (m)	0.8 – 1.8	(Osorio and Hontoria, 2001;
	3 – 4	M'Coy, 1997)
Contact Time (hr)	0.5 – 1	(M'Coy, 1997;
	1 – 1.5	Kinner and Eighmy, 1989)
Backwash		
Interval (hr)	24 – 48	(M'Coy, 1997; Stepenson et al., 1993)
Length (hr)	0.33 – 0.66	(M'Coy, 1997)
Water Requirements	≤20% of influent flow	(M'Coy, 1997)
Maximum Headloss	2 m	(M'Coy, 1997)

Ranges of typical design and operational variables for BAF technology in general are given above, in Table 2-6, and performance is given in Table 2-7, below. It should be noted that some BAFs have been reported to achieve suspended solids effluents as low as 10 mg/L (M'Coy, 1997).

Table 2-7. Typical BAF Performance (after M'Coy, 1997).

Parameter	Typical Loading Rate	Typical Effluent Quality
Chemical Oxygen Demand (COD)	< 6 kg $COD/m^3/d$	< 90 mg/L
Total Suspended Solids (TSS)	< 3 kg $TSS/m^3/d$	< 30 mg/L
Total Kjeldahl Nitrogen (TKN)	< 0.45 kg $TKN/m^3/d$ (high C/N) < 1.5 kg $TKN/m^3/d$ (low C/N)	< 5 mg/L ammonia < 5 mg/L ammonia
Hydraulic	48 to 144 $m^3/m^2.d$ (high C/N) 144 to 360 $m^3/m^2.d$ (low C/N)	—

* where C/N is the carbon to nitrogen ratio

Consolidation Strategy

Bioclarifiers, such as the ones previously discussed, conform to the ideals of the consolidation strategy. The consolidation strategy is a means of reducing the overall number of units and processes required in a treatment train by combining multiple functionalities into single structures. As worldwide populations increase and cities experience rapid growth, additional demands will be exerted on the wastewater treatment industry to provide technologies capable of reducing environmental impact while increasing economic efficiency. By incorporating the ideals of the consolidation strategy into wastewater treatment designs, complex treatment trains may be reduced into a fewer number of units that conform to the economic realities and spatial requirements confronting today's communities. While the resulting units may not be optimized to treat each individual contaminant, the units can be designed to sufficiently treat wastewater to meet increasingly stricter effluent requirements. Savings in capital and operating costs for single components serving multiple duties may overcome the disadvantages of decreased individual processes efficiencies (Loyless and Malone, 1998).

The attributes of simplicity, efficiency, and flexibility, which are key to the consolidation strategy, are generally present in bioclarifier design. The great advantage of the bioclarifier process is that it can serve as both a bioreactor and as a solid-liquid separation device simultaneously, eliminating the need of a clarifier. The high biomass concentration and solids separation capacity of bioclarifiers provides a unique combination of long sludge age (SRT) and short hydraulic retention time (HRT), which is favorable for minimal land usage for wastewater treatment facilities (Chen *et al*., 2000).

Static Low-Density Media Filters

Static low-density media (SLDM) filters are known in the aquaculture community as Floating Bead Filters (FBFs) or Floating Bead Bioclarifiers (FBBs). Use of the SLDM filter dates from the mid-1970's, from which time the filters have evolved through several design iterations to improve bioclarification capabilities. A concise history may be found in Wu (2003). The units are currently widely employed as clarifiers or bioclarifiers in support of high-density recirculating production and holding systems for fish, reptiles, and crustacea, (Malone and Beecher, 2000; DeLosReyes and Lawson, 1996). Historically, SLDM filters have been used exclusively in aquaculture applications; therefore, most research has been in improving nitrification capacity, as that has been identified as the limiting factor in bioclarifier performance (Sastry, 1999; Malone *et al*., 1993). SLDM filter technology has not been widely applied to domestic wastewater for concurrent biological and physical treatment. Technologies that have been employed in domestic wastewater treatment capacities similar to SLDM filters include: biological aerated filters (BAFs), moving bed biofilm reactors (MBBRs), and sand filters.

SLDM filters are fixed-film filters, typically operated in an upflow configuration, in which a biofilm support medium is submerged in wastewater to create a large contact area for aerobic biological treatment. Biofilm carriers used in SLDM filters have a specific gravity of less than one, which results in a floating bed of filter media. Dissolved oxygen is supplied hydraulically to the filter without disturbing the media, resulting in a static bed and typically necessitating recirculation of the wastewater in order to maintain aerobic conditions. Water pumps are typically used for recirculation through the bioclarifier bed, aeration is then provided by diffusers or aeration trays. In keeping with the consolidation strategy, an alternate mechanism for combined water movement and aeration, namely airlift pumps, may be used. Airlift pumps have been found to readily provide dissolved oxygen to submerged packed beds challenged with elevated BOD levels (Loyless and Malone 1998; Reinemann and Timmons, 1989).

Figure 2-3. Various shapes of plastic media have been tested in SLDM Filters in the past. From top to bottom: KMT-type, large tubes, smaller tubes, Enhanced Nitrification (EN) modified, and spheres.

The low-density plastic media, as seen in Figure 2-3, acts as a carrier for biofilm and as a physical separation mechanism for solids. Heterotrophic bacteria attach themselves to the beads and utilize the organic matter in the waste stream as a carbon source for growth, while autotrophic, nitrifying bacteria convert ammonia to nitrate under conditions of low organic loading (Zhang et al, 1995). Concurrently, suspended solids in the waste stream are captured in the bed via surficial straining, deep bed filtration, and adsorption as the waste stream travels upward though the bead bed. The capture of solids in a SLDM filter is known to be influenced by particle size, filter media size, flowrate, and solids accumulation (Ahmed, 1996). Presence of biofilm on media has also been shown to positively influence solids capture in floating granular medium filters (Yoo and Kim, 2001).

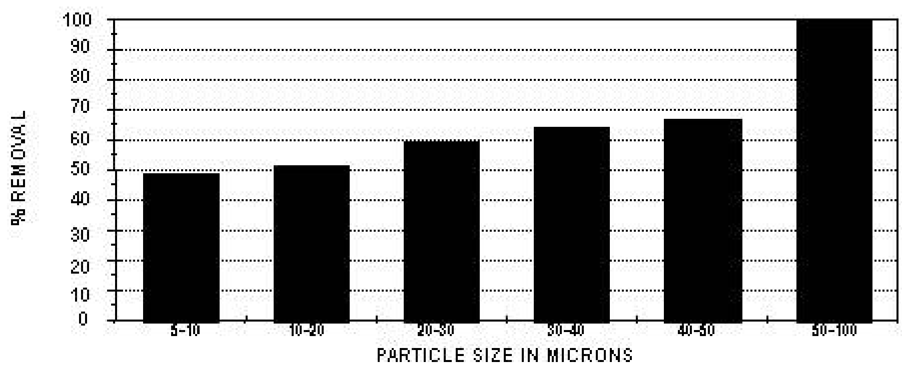


Figure 2-4. Percent removal versus particle size capture in a RAS using a Propeller-Washed Bead Filter for biofiltration and solids capture (from Ahmed 1996). The filter contained 3 mm spherical plastic beads with biofilm development.

The solids capture mechanisms of floating media filters are largely physical in nature and are common to all types of granular medium filters. Previous studies on SLDM technology have shown that acclimated filters, that is, media with an established biofilm as it would be in a wastewater treatment application, remove nearly 100 percent of the suspended particles larger than fifty microns in diameter on a single pass through the filter bed; whereas, single pass removal efficiency for unacclimated filters drops to about 20 percent for particles below 10 microns. Development of biofilm improves fine solids capture in a single pass up to 48 percent for particles below 10 microns (Ahmed, 1996). Recirculating systems are capable of more complete solids removal, where fine solids are removed progressively through multiple passes through the filter bed.

As SLDM filters were developed for the recirculating aquaculture industry, the filters were designed to operate with aeration occurring externally from the filter. Recirculating aquaculture systems use water movement as a means of facilitating removal of waste products via mixing and mass transport of solids and substrate (Loyless and Malone, 1998). Recirculation of wastewater through the bioclarifier bed may have developed as an artifact of SLDM filter use in aquaculture; however, the potential benefit of increased solids removal through a static bed has resulted in the continuation of the practice of aerating the biofilm carriers externally. The static nature of the media bed remains as one of the distinguishing characteristics of SLDM filters.

External aeration is another major differentiating point between SLDM filters and other aerobic fixed bed technologies, which use internal aeration and long residence times inside of the biofilters. When airlift pumps are used to provide external aeration, multiple passes are made through the filter bed with retention times of 30 seconds to 1.5 minutes per pass (Wagener *et al.*, 2002). At the end of each pass, the wastewater must return to a recirculation basin. Longer one-pass retention times may be obtained, however unless additional oxygen is added, a portion of the filter bed will likely become anaerobic. The total hydraulic retention time (HRT) inside of the filter bed can be calculated by multiplying the one-pass retention time by the total number of passes through the filter.

One drawback to granular medium biofilters, particularly submerged granular medium biofilters, is the build up of headloss in the biofilm carrier (Ødegaard *et al.*, 1994). Backwashing is typically the means for reducing headloss, however; backwashing can result in substantial loss of water, require a large energy input, and can halt treatment. Technologies have been developed in an effort to overcome such disadvantages of backwashing and are used in SLDM filters. These include backwashing with propellers or with air, along with selection of floating media. Figure 2-5 illustrates both the filtration and the backwashing modes of a propeller-washed unit, while Figure 2-6 illustrates the two modes in one design of a pneumatically washed unit.

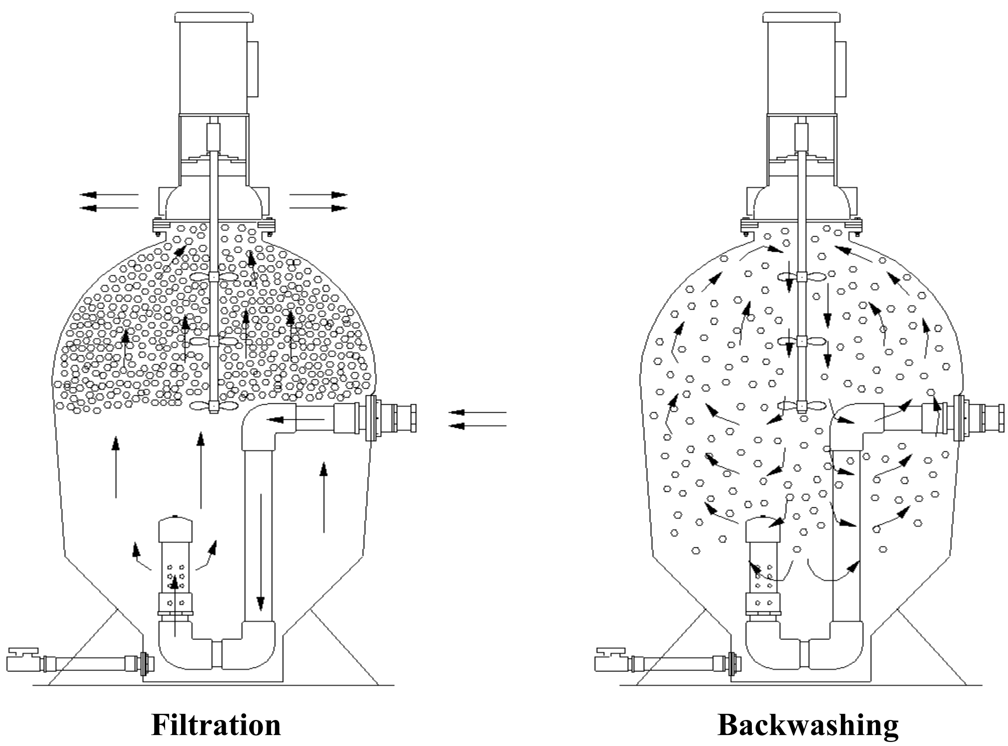


Figure 2-5. A propeller-washed SLDM filter is backwashed by first expanding the bed with a propeller then allowing the sludge to settle as the bed reforms by flotation.

The beds of SLDM filters are periodically expanded for removal of accumulated solids and excess biofilm (Malone and Beecher, 2000; Cooley, 1979). Backwashing, or expansion, of a filter bed can be accomplished by hydraulic, pneumatic, or mechanical means. SLDM Filters take advantage of the buoyancy of the media to minimize the water loss that would otherwise be associated with the high frequency washing needed to manage the biofilm. These units are capable of restricting water losses to periods of sludge removal, as opposed to other filters, which can use ten percent of the product water (Stensel *et al.,* 1988) to twenty percent of the influent wastewater (M'Coy, 1997) to hydraulically wash the media.

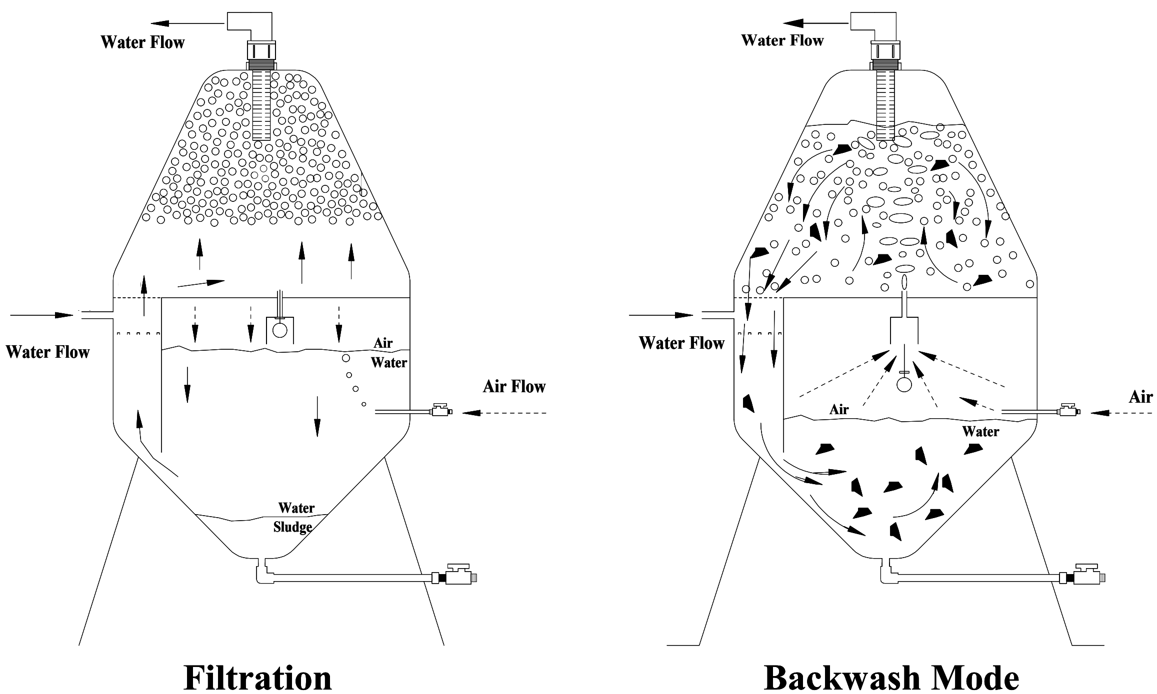


Figure 2-6. In an air washed SLDM Filter the media experiences a gentler backwash, and a higher backwashing frequency may be used.

During normal operation, SLDM filters operate with a packed bed, regardless of backwashing technique. Filtration is periodically disrupted during periods of backwashing, in which the media is expanded and captured solids along with generated biomass is allowed to escape the media bed. The sludge then settles in the bottom of the SLDM filter hull, where it is stored and from which it is discharged.

These units are virtually impervious to caking problems that can plague granular filters subject to high organic loads. Since backwash water loss is minimal in both hydraulically and mechanically washed filters, backwash frequency may be employed as a biofilm management tool (Malone *et al.*, 1993). Additional biofilm management flexibility is obtained by altering the bead shape or by moving to a less aggressive washing format (Golz *et al.*, 1999). However, once the unit is selected, backwash frequency is the principal operation parameter used to enhance biofiltration performance.

Low-density media filters have been studied in the past in various configurations, specifically in Western Europe and Japan, with a focus on their ability to perform: particle separation alone in a SLDM filter (Miyaki *et al*., 2000) and separation enhanced with coagulation in a static bed (Liao and Ødegaard, 2002; Ødegaard, 1998; Visvanathan *et al*., 1996; Tanaka *et al*., 1995); biofiltration of domestic wastewater alone in a dynamic filter for secondary (Andreottola *et al.*, 2000; Ødegaard *et al*, 1994) and tertiary (Maurer *et al*., 2001) treatment; or bioclarification with internal aeration for secondary (Yoo and Kim, 2001; Mann *et al*., 1995) and tertiary (Payraudeau *et al.*, 2000; Mann *et al.*, 1998; Boller *et al.*, 1997) treatment. The performance of a SLDM filters for concurrent BOD and TSS removal in the domestic wastewater arena has yet to be established.

SLDM Filter Classification

Research and development in fixed film technology, specifically aerobic high rate biological fixed film processes, has been a rapidly expanding field in the last two decades. Classification of the various fixed film processes may be made based on a variety of characteristics, such as submergence, aeration technique, and expansion state of the media, as shown in Figure 2-7. Static Low Density Media filters are distinguished from other biofiltration processes by their external as opposed to internal aeration strategy, their static media matrix, and by their ability to function as a bioclarifier. An additional characteristic of SLDM filters is its ability to function as an aerobic unit, an anaerobic unit, or a combination of the two. Long one pass residence times inside of internally aerated media filter beds can result in anaerobic conditions in a portion of the bed, which may be desirable if both aerobic and anaerobic process are needed. A brief description of the fixed film processes similar to SLDM filters is presented.

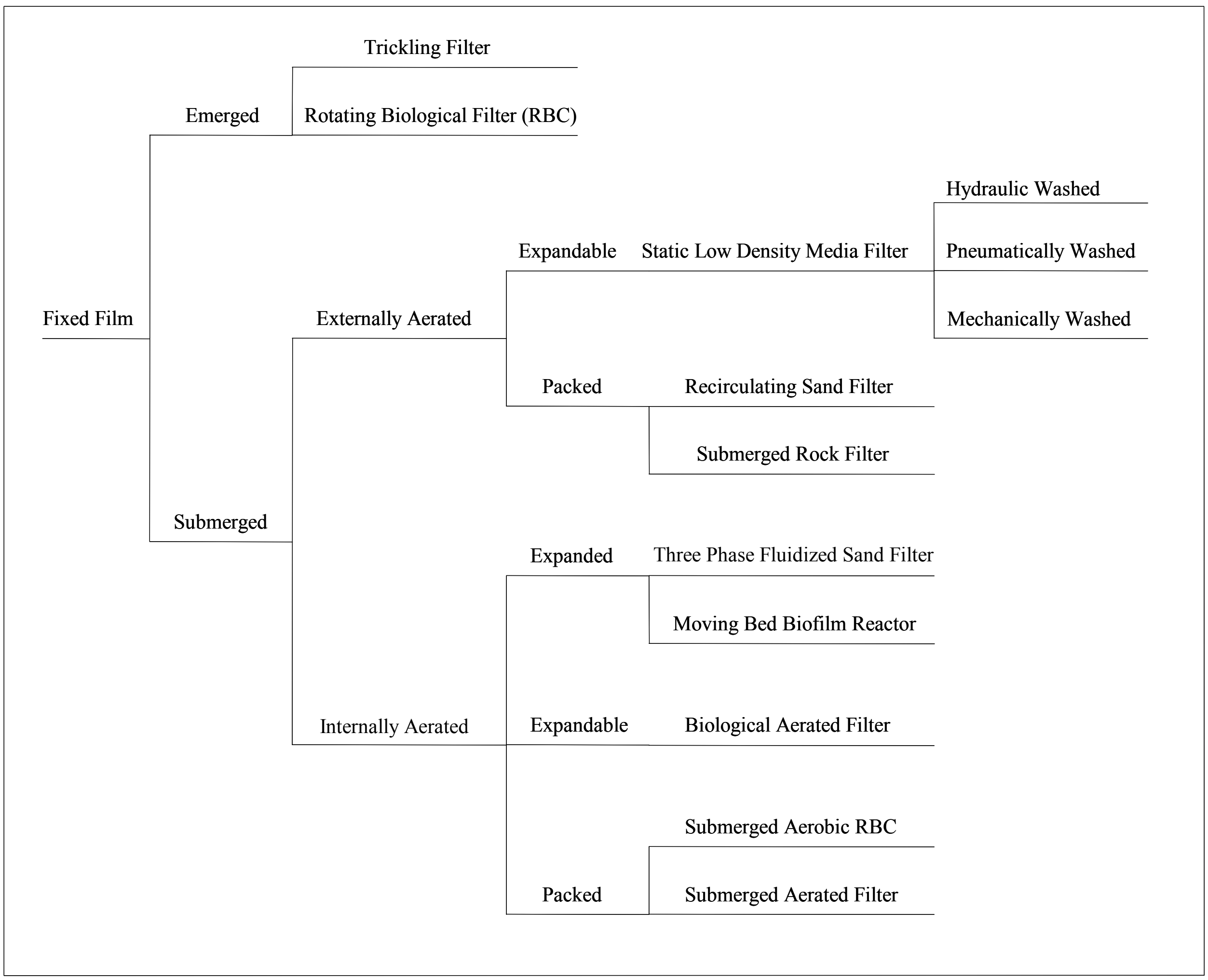


Figure 2-7. Classification of various major aerobic fixed film processes used in wastewater treatment.

Aerobic fixed film processes may be divided into two categories: emerged or submerged. The media in emerged processes, such as trickling filters and rotating biological contactors (RBCs), are intermittently in contact with air, eliminating the need for supplementary aeration. Media in submerged processes are fully enveloped by wastewater, thus requiring additional aeration to maintain aerobic conditions. Aeration in submerged fixed film processes may occur either externally or internally. Variations in aeration configuration may not affect the removal of soluble organic material in the media, but solids capture, biofilm structure, and microbial populations may be impacted. Superficial air velocity has been found to be a controlling factor of biofilm morphology; increasing detachment forces results in thinner and denser biofilms (Liu and Tay, 2001; Tijhuis *et al*., 1996). The response of biofilm to additional stress is not only physical, but also results in decreased observed growth yield (Y_{obs}) and metabolic responses (Liu and Tay, 2001). Static media beds, as achieved in external aeration, should be capable of higher solids retention rates than the pulsed or fully dynamic media beds of internally aerated biofilters.

Fixed film processes may be further delineated by classifying the physical state of the media bed. Beds may either be expanded, expandable, or packed. During periods of rest, inert media will occupy a certain, fixed volume. By fluidizing the media, an expanded state may be achieved. By example, moving bed biofilm reactors (MBBR) have a filling ratio of 60 to 70 percent of the reactor volume; when operated, the interstitial spaces of the media expand so that the media fills the entire reactor volume. Expandable beds are run in two separate modes – expanded during backwashing and packed during filtration mode. Packed beds are never expanded, and are typically constructed of high-density media or fixed, block media.

Objectives

The objectives of this study are to answer a variety of questions related to the applicability and performance of Static Low-Density Media Filters when treating domestic wastewater.

(1) At what levels of loading – organic and hydraulic – can various configurations of SLDM filter prototypes achieve self-imposed effluent concentration limits of 10/10 $CBOD_5$/TSS?

(2) What type of filter/recirculating tank configuration of those evaluated will provide optimal treatment?

(3) How do SLDM filters compare with other similar technologies and more traditionally used technologies for the treatment of domestic wastewater?

CHAPTER 3 : STATIC LOW-DENSITY MEDIA FILTER FOR REMOVAL OF SOLIDS AND ORGANICS FROM A VARIABLE FLOW DOMESTIC WASTEWATER SOURCE

Introduction

The core of most domestic wastewater treatment plants is defined by a classical unit operation and process approach. Treatment plants designed using the unit operation technique consist of individually optimized units and processes in the typical sequence of primary clarification followed by biological treatment and then secondary clarification. These serial treatment steps are used to: remove solids from the wastewater influent, remove dissolved organic materials via biological activity, and then physically remove the biomass generated during biological treatment. While this approach is technically sound and effective, it results in treatment plants requiring a large footprint, substantial capital investments, and sophisticated staffs. Meanwhile, the demands placed on the wastewater treatment industry are for technologies that are robust and relatively simple to operate without any compromise of effluent quality. Systematic consolidation of the treatment train through utilization of technologies that can perform more than one process or operation may simplify complex treatment trains while providing additional economic and footprint advantages.

Units capable of simultaneous physical and biological treatment have been developed and typically employ a granular medium. While granular packed bed filters have traditionally been used solely for solid-liquid separation, their natural propensity for biofiltration along with clarification has been observed. To capitalize on this biofiltration capacity, granular medium has been increasingly used as a biofilm carrier in fixed film bioreactors. These biofilters have generated interest in the last few decades due to their smaller reactor size, improved removal efficiency, and robustness against shock loadings as compared to conventional treatment methods. Some granular medium biofilters have solids removal capability incorporated into biofilter design, which enables operation of the unit without a downstream clarifier. The resulting consolidated unit may be referred to as a bioclarifier, an apparatus that not only uses a physical medium as a biofilm carrier for fixed film growth, but also simultaneously uses the filtration capacity of the media for its solid-liquid separation abilities. Bioclarifiers currently in use in domestic wastewater treatment include Biological Aerated Filters (BAFs) and some on-site recirculating sand filters.

A variety of granular media have been used in bioclarifiers, although increasingly this media has been formed from a variety of plastics. Extrusion processes have allowed greater control over size, shape, and density of plastic biofilm carriers, which result in greater potential for operational control and optimization (Mann *et al*., 1995). The media factors influence the effective and available biofilm surface area for growth, solids capture efficiency, backwashing requirements and frequency, quantity of producible product water, and hydrodynamics within the bioclarifier. Specific gravity in particular has proven to be a factor that greatly influences bioclarifier performance. Floating medium filters have been shown to have higher solids retention capacities and lower headloss development when compared to conventional higher-density medium filters (Ngo and Vigneswaran, 1995), along with low head loss during filtration and reduced energy requirements for backwashing (Tanaka *et al.*, 1995). Floating media have also been shown to outperform sunken media for COD and ammonia removal in addition to TSS removal (Mann *et al*., 1998).

This paper describes the Static Low-Density Media (SLDM) filter, a submerged granular medium externally aerated bioclarifier, and reports on early findings related to performance of bench scale SLDM units used to treat domestic wastewater. Low-density media filters have been studied in the past in various configurations, specifically in Western Europe and Japan, with a focus on their ability to perform: particle separation alone in a SLDM filter (Miyaki *et al*., 2000) and separation enhanced with coagulation in a static bed (Liao and Ødegaard, 2002; Ødegaard, 1998; Visvanathan *et al*., 1996; Tanaka *et al*., 1995); biofiltration of domestic wastewater alone in a dynamic, moving bed filter for secondary (Andreottola *et al.*, 2000; Ødegaard *et al*, 1994) and tertiary (Maurer *et al*., 2001; Hem *et al*. 1994) treatment; or bioclarification with internal aeration for secondary (Belgiorno *et al*., 2003; Yoo and Kim, 2001; Mann *et al*., 1995) and tertiary (Payraudeau *et al.*, 2000; Mann *et al.*, 1998; Boller *et al.*, 1997) treatment. The performance of SLDM filters for concurrent BOD and TSS removal in the domestic wastewater arena has yet to be established.

Static Low-Density Media Filters

Some classes of Static Low-Density Media (SLDM) filters are known in the aquaculture community as Floating Bead Filters (FBFs) or Floating Bead Bioclarifiers (FBBs). Use of the SLDM filter dates from the mid-1970's, from which time the filters have evolved through several design iterations to improve bioclarification capabilities. A concise history may be found in Wu (2003). SLDM filters, such as the one shown in Figure 3-1, are currently widely employed as either clarifiers or bioclarifiers in support of high-density

Figure 3-1. Static Low Density Media (SLDM) filters employ granular floating medium for concurrent solids capture and biological filtration.

recirculating production and holding systems for fish, reptiles, and crustacea (Malone and Beecher, 2000; DeLosReyes and Lawson, 1996). Historically, SLDM filters have been used exclusively in aquaculture applications; therefore, most research has been in improving nitrification capacity, as that has been identified as the limiting factor in bioclarifier performance for aquaculture applications (Sastry, 1999; Malone *et al*., 1993). SLDM Filter technology has not been widely applied to domestic wastewater treatment for concurrent biological and physical treatment. Technologies similar to SLDM filters that have been employed in domestic wastewater treatment capacities include: biological aerated filters (BAF), moving bed biofilm reactors (MBBR) and sand filters.

SLDM filters are fixed film filters in which a biofilm support medium, characterized as both granular and floating, is submerged in wastewater to create a large contact area for aerobic biological treatment. In the packed bioclarification mode, the unit concurrently provide solids capture, carbonaceous BOD removal, and under conducive conditions, nitrification. The low-density plastic medium acts as a carrier for biofilm and as a physical separation mechanism for solids. Chemoheterotrophic bacteria attach themselves to the granular media and utilize the organic matter in the waste stream as a carbon source for growth, while autotrophic nitrifying bacteria may convert ammonia to nitrate under conditions of low organic loading (Zhang *et al*., 1995). Concurrently, suspended solids in the waste stream are captured in the bed via surficial straining, sedimentation, deep bed filtration, flocculation, and adsorption as the waste stream travels upward though the bed (Malone and Beecher, 2000).

The units are normally operated in an upflow configuration, with the floating bed in a packed or static mode. Periodically the beds of SLDM filters are dynamically expanded via hydraulic, pneumatic, or mechanical means to remove accumulated solids and excess biofilm

(Malone and Beecher, 2000; Cooley, 1979). The disadvantages of backwashing, such as substantial loss of water, large energy input requirements, and halting of treatment have been largely overcome by various backwashing mechanisms developed for use in SLDM filters. The buoyancy of the media used in SLDM filters provides the advantage of minimizing water loss that would otherwise be associated with the high frequency washing needed to reduce headloss in the filter. In addition, mechanically and pneumatically washed units are capable of restricting water losses to periods of sludge removal, as opposed to other filters that can use ten percent of the product water (Stensel *et al.,* 1988) to twenty percent of the influent wastewater (M'Coy, 1997) to hydraulically wash the media.

The two operational modes of water conserving, pneumatically washed units are illustrated in Figure 3-2. During the filtration mode, influent water enters the filter below the

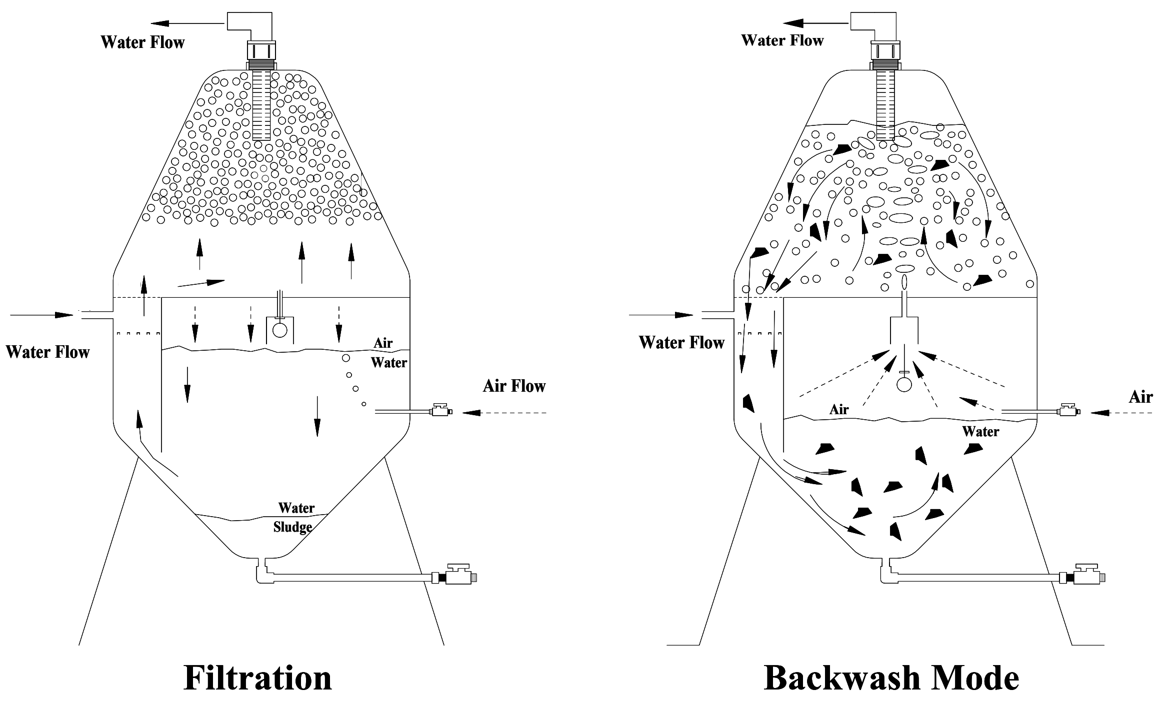


Figure 3-2. The two modes of operation in a SLDM filters are filtration and backwashing. In the type of SLDM filter shown above, air was used to backwash the floating media, which minimized water losses.

media bed and exits from the screened outlet above the media bed, simultaneously as air enters the charge chamber. The charge chamber is an airtight compartment below the filter bed that stores the air used for backwashing. Once the charge chamber has reached its volumetric capacity, a pneumatic trigger fires and the backwashing mode commences. During the backwash mode, which typically lasts for less than two minutes, air from the charge chamber is released into the filter bed, agitating the media. This action concurrently shears accumulated solids and excess biofilm from the media and interstitials of the bed, and forces backwash water into the bottom of the charge chamber, which acts as a sludge storage area. It is important to note that during backwashing neither air nor wastewater application to the filter is interrupted. Once the charge chamber ceases to release air, the media bed reforms and floats upwards and the filter reenters its filtration mode. As the solids in the backwash water settle, the supernatant is again passed through the filter bed, while the air chamber is recharged. Sludge may be drained manually or via control systems from the sludge outlet in periodic intervals set by the operator.

Backwashing mechanism, intensity, and frequency are parameters that can be used to control the concentration and morphology of the microbial consortia in a biofilm reactor. Since backwash water loss is minimal in SLDM filters, backwash frequency may be employed as a practical biofilm management tool (Malone *et al*., 1993). Additional biofilm management flexibility may be obtained by altering the media size and shape, although once the unit is selected, the principal operation parameter used to enhance biofiltration performance is backwash frequency (Golz *et al*. 1999).

Aerobically operating SLDM filters are aerated externally from the media bed. Dissolved oxygen is supplied hydraulically to the filter via the influent wastewater, which minimizes disturbance of the media and results in a static bed. This necessitates recirculation of the wastewater in order to maintain aerobic conditions within the bed. Water pumps are typically used for recirculation through the bioclarifier bed and aeration may then be provided by aeration diffusers or trays. To further consolidate the number of treatment units, an alternate mechanism that combines water movement with aeration, namely airlift pumps, may be used. Airlift pumps have been found to readily provide dissolved oxygen to submerged packed beds challenged with elevated BOD levels (Loyless and Malone 1998; Reinemann and Timmons, 1989). An effective treatment strategy with the functions of recirculation, aeration, biofiltration, and secondary clarification results from the combination of an air blower, recirculating tank, and a SLDM filter.

Materials and Methods

The performance of two experimental designs of Static Low-Density Media (SLDM) filters fed with a primary treatment effluent of domestic wastewater from an industrial plant was evaluated. The experimental systems (BF4 and BF6) were operated at ambient conditions, while operational parameters such as backwashing frequency, filtration rate, and total daily flow were controlled. Both systems consisted of a submerged upflow SLDM filter in conjunction with a constant volume-equalization tank used also for recirculation. For all studies, influent and effluent samples from the SLDM filter-equalization tank system were collected, as well as samples from immediately prior to and immediately after traversing the media bed.

Experimental systems BF4 and BF6 were evaluated in this study for a total period of nine months. Both systems ran in an acclimation mode for a period of at least one month prior to analytical testing. During this period wastewater was circulated through the filter, but the backwashing frequency was lowered so bacteria could populate the biofilm carriers. The biofilm carrier throughout the entire research effort was enhanced nitrification (EN) modified media, 3 to

5 mm in length with a density of 900 kg/m^3, a clean bed porosity of 0.55, and with a total specific surface area (SSA) of approximately 1100 to 1250 m^2/m^3 (Malone *et al*., 1993). Figure 3-3 illustrates the shape of EN media. Both filters operated at filtration rates between 5 and 30 m/h, classified as high filtration rates by Ødegaard and Helness (1999). Propeller-type flow meters were installed on the tail end of each system, which gave accumulative volume measurements from which daily flows were calculated. Periodically, a few gallons of, sludge were discharged from the filters typically once per week. Descriptions of the site characteristics and the experimental setup and methodologies of each experimental system follow.

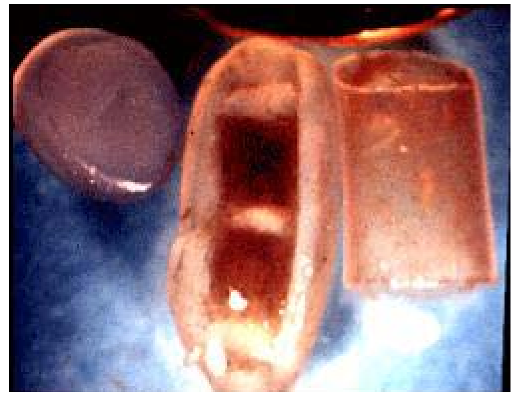


Figure 3-3. Various shapes of low-density media have been tested in SLDM filters. From left to right, spherical, EN, and tube media can be seen with biofilm deposits.

SITE CHARACTERISTICS

The experimental units were in operation at an outdoor facility that received domestic wastewater from approximately 40 employees at an industrial plant in Denham Springs, Louisiana, USA. The site was subject to highly variable flow characterized by morning and afternoon peaks, no overnight flow, and no weekend flow. Raw wastewater from the facility was intermittently pumped from a below ground sump into a 3.79 m^3 (1000 gallon) fiberglass tank that functioned as both a storage tank and also as a primary clarifier, however not as a flow equalization basin. Influent to the experimental systems was gravity fed directly from this tank. Flow delivery to the experimental units was proportional to the rate of wastewater generation by the industrial facility. Since the units were fed with real wastewaters, uniform BOD and TSS concentrations were not achieved in all phases of the study. An attempt was made to cover a wide range of applied hydraulic and organic loads. The temperature of the wastewater in the filter bed varied based on ambient conditions throughout evaluation from 7 to 32°C, which allowed the opportunity to observe the effect of temperature on treatment performance over the range of organic loadings used. The mean influent wastewater characteristics for experimental units BF4 and BF6 can be found below in Table 3-1.

Table 3-1. Characteristics of the wastewater entering the experimental systems.

	BF4	BF6
$CBOD_5$, mg/L	104 ± 23.4	145 ± 29.1
(n)	(33)	(25)
TSS, mg/L	60 ±21.3	91 ±39.2
(n)	(20)	(22)
Temperature C	22.6 ± 6.7	17.7 ±4.0
(n)	(37)	(25)

* (n) indicates the number of sampling events
**The standard deviation follows the ± sign

EXPERIMENTAL SYSTEM BF4

Experimental system BF4 was in operation from June 2001 to January 2002. The major system components were a peak flow mitigation basin (also referred to as a recirculation tank), a SLDM filter, and an airlift pump used for recirculation and reaeration. The general configuration of the BF4 system is shown in Figure 3-4. Since the influent flow to the system was not equalized and generally devoid of dissolved oxygen, a T-connector was used to allow the influent wastewater to enter directly into the filter in combination with flow from the recirculation tank. This increased dissolved oxygen levels and provided consistent flow to the filter even during periods of low or no flow of influent.

The recirculation tank for this system was a 1.42 m^3 fiberglass tank connected to the SLDM filter. The filter was made out of a 1.22 m (4 ft) tall, 0.61 m (2 ft) diameter section of PVC pipe and contained 42.5L (1.5 ft^3) of floating EN media with a media depth of 30.5 cm.

Screens contained the filter media on the top of and below the filter bed. The air charge chamber and the release trigger rested below the bottom screen. An air pump was used to introduce air into the airtight charge chamber, which served a dual function as a sludge storage area.

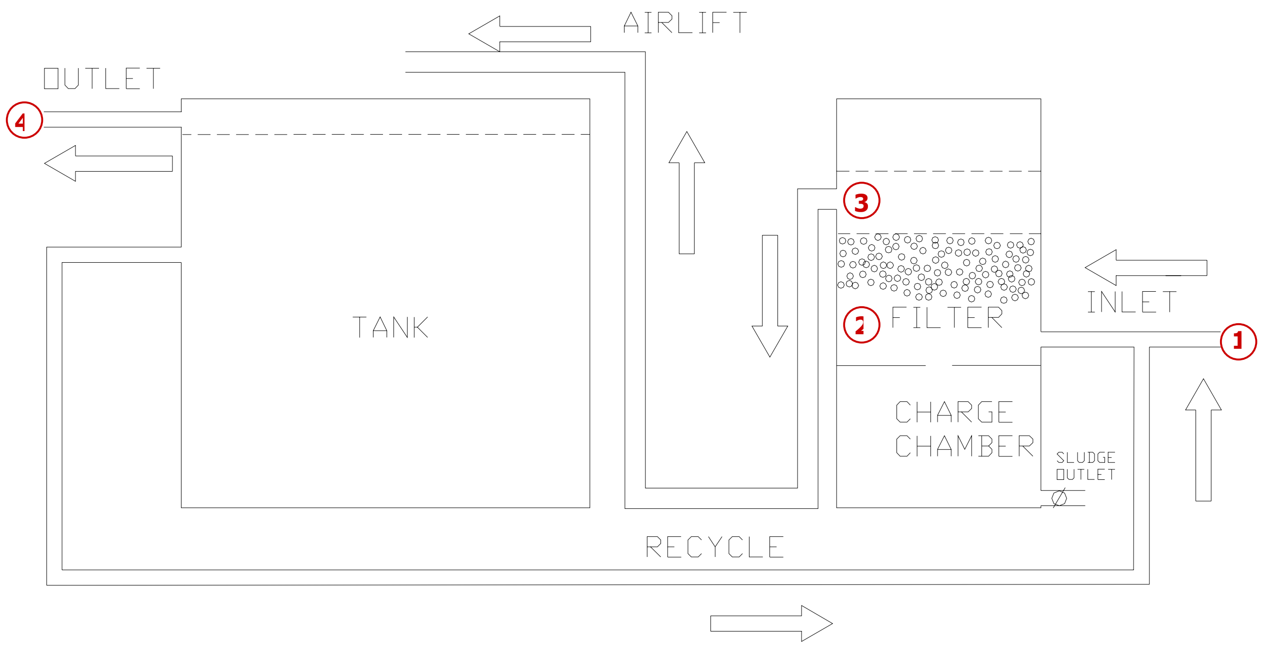


Figure 3-4. Experimental system BF4 consisted of a SLDM filter, recirculation tank, and an airlift pump. Sampling port locations are indicated by circles.

In the BF4 system, a concentric airlift pump was used; the downdraft pipes for water flow and for air delivery traversed the internal areas of the SLDM Filter. The airlift pump was used to return the wastewater from the top of the filter to the top of the equalization basin. Once in the equalization basin, the flow was mixed and returned to the SLDM filter. Discharge from the system occurred during periods of overflow from the recirculation tank, into an effluent holding tank. System effluent samples were taken from the discharge line into this sump. The effluent sump contained a trash pump and was followed by a meter so that the total volume exiting through the system could be determined. Figure 3-5, a photograph of the BF4 system, further illustrates the layout. As shown in the picture, the recirculation tank was kept covered to minimize algae growth and evaporation.

Figure 3-5. A photograph of the BF4 system during operation at the outdoor testing facility. The recirculation tank was kept covered.

Experimental System BF6

After observing the performance of BF4, design modifications were implemented and a new system was designed and fabricated. Experimental system BF6 was in operation from November 2001 to April 2002. As in the BF4 system, the major system components were a constant volume equalization basin, a SLDM filter, and an airlift pump, and the system was fed from the same 1000-gallon fiberglass primary settling tank. The BF6 unit had a total capacity of 1.78 m^3 (four feet in diameter and five feet tall) and was constructed out of fiberglass. In contrast with the BF4 system, the SLDM filter in BF6 was placed in the middle of the equalization basin, and a concentric partition, three feet in diameter, was also placed in this tank. The resulting effect was one tank with three separate chambers: an outside atrium chamber; an inside chamber;

and the filter bed. Influent wastewater was added to the outer chamber of the equalization basin vertically at the water surface of the tank. Perforations facilitated water movement between the outside atrium and the inner chamber. These exchanges were driven for the most part by the expansion and collapse of the air charge associated with the backwash mechanism.

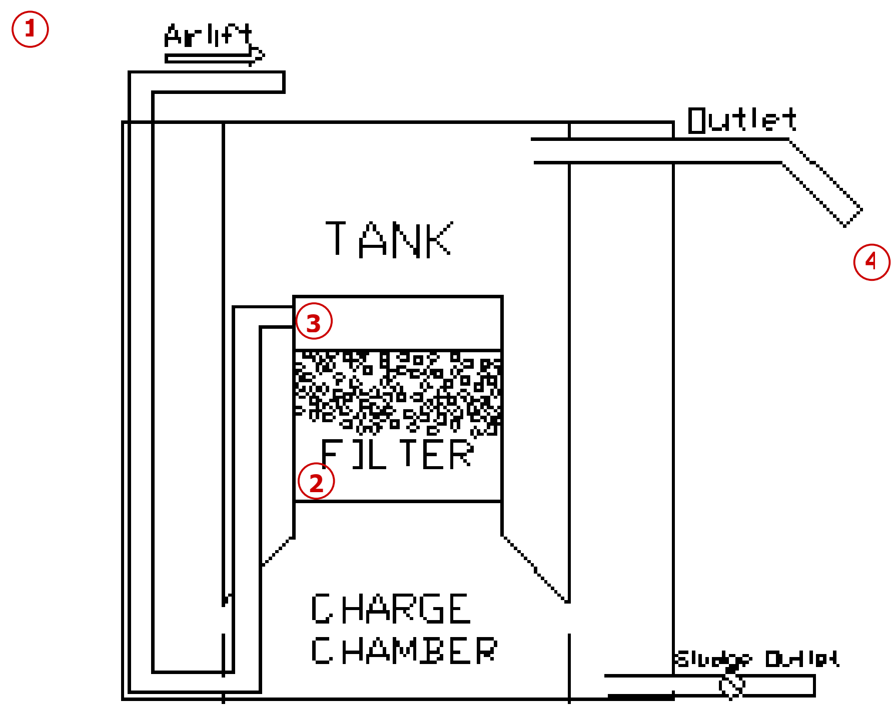


Figure 3-6. Experimental system BF6 was based on a concentric design with the same three components used in system BF4. Not shown in this drawing is a second airlift pump.

The filter contained 113.3L (4 ft^3) bed of EN biofilm carriers, 38.1 cm in depth. The media were kept separated from the bulk liquid in the tank by a fiberglass chamber, and contained by top and bottom screens. Sampling ports were located above the top screen and above the bottom screen. Airlift pumps were used to steady the direction of flow and circulate water from the filter to water surface of the inner chamber, where it was allowed to recirculate through the filter bed. In Figure 3-6, only one airlift is seen, although the system was equipped

Figure 3-7. A photograph of BF6 system illustrates the concentric design. The filter had recently experienced a backwash. The media was contained in a fiberglass hull below the water surface.

with two airlift pumps located on a single axis that traversed through the center of the filter. Following the filter-tank combination was an effluent holding tank. This effluent sump contained a trash pump and was followed by a meter so that the total volume exiting through the system could be determined. The effluent traveling to this sump was used as the system effluent sample during the study. Sampling sites were similar to the BF4 system. Specific operating detail of experimental system BF6 may be found in Appendix A.

Sampling and Measurement

Grab samples were taken from four separate sampling locations in both experimental systems: immediately prior to and after traversing the media bed, referred to as recycle in and out respectively, from the source influent to the system (system in), and from the effluent of the system (system out). Specific sampling procedures may be found in Appendix A.

Analytical Methods

Temperature, pH, daily flow, and recirculation flow measurements were recorded along with other operational parameters, such as backwash frequency, during each sampling event. Water quality parameters were tested in triplicate according to Standard Methods and included: $CBOD_5$ (5210B), DO (4500-O C), TSS (2540 D), and VSS (2540 E) (APHA, 1995). Dissolved oxygen samples were preserved immediately after collection and were brought to the Water Quality Laboratory at LSU, along with the other samples, for analysis.

Results and Discussion

Experimental results for the BF4 evaluation period from July 2001 to January 2002 have shown carbonaceous biochemical oxygen demand ($CBOD_5$) concentrations decreasing from 104 mg/L to 9 mg/L (and in some cases, to less than 3 mg/L) on average for this period and through multiple passes of the filter. Total suspended solids (TSS) concentrations were shown to decrease from 60 mg/L to 9 mg/L on the average.

The BF6 unit was evaluated from December 2001 to April 2002. The average $CBOD_5$ concentrations decreased from 145 mg/L to 25 mg/L and the TSS decreased from 91 mg/L to 32 mg/L. This also was for multiple passes through the filter and operated under various regimes. Table 3-2 details the results for the various operational series.

Volumetric Loadings Applied to the Reactors

The volumetric loading of $CBOD_5$ applied to the reactors is shown in Figure 3-8. The use of recirculation to sufficiently aerate the biologically active beds of SLDM filters has resulted in two different organic loading parameters: total system loading and bed loading. Methods used to calculate values for these two parameters are given in Equations 3.1 and 3.2.

$$Loading_{Total} = \frac{S_{Si} * Q}{V_b} \qquad \text{Equation 3.1}$$

Where S_{Si} is the substrate concentration of either $CBOD_5$ in the system influent in g/m^3, Q is the daily flow rate through the entire system in m^3/d, and V_b is the volume of media in the filter bed and not the filter hull volume, in L. The media was considered to be the functional volume of the systems, wherein the biological conversions and physical separation occurred; additional volume occupied by the systems were considered to function solely as peak flow mitigation. Bed loading, as opposed to the total system loading, incorporates Q_r, the recirculation (or recycle) flow rate, which is the flow rate approaching the filter bed. The expression used to calculate volumetric bed loading has similar for as the system loading rate, and is given in Equation 3.2.

$$Loading_{bed} = \frac{S_{ri} * Q_r}{V_b} \qquad \text{Equation 3.2}$$

Where S_{ri} is the concentration of $CBOD_5$ or TSS immediately prior to entering the filter media bed (recycle in) in g/m^3, Q_r is the flow rate through the filter, accounting for the increased effects of recirculation, in m^3/d and V_b in L, as previously defined.

Due to daily variations in wastewater generation at the facility, it was difficult to obtain stable loads for any considerable length of time. The volumetric organic load applied to the bed of reactor BF4 varied in the range of 2.5 – 34.6 $kg/m^3.d$, while BF6 had an organic bed loading range of 2.8 – 56.9 $kg/m^3.d$. Volumetric organic loadings to the system ranged from 0.3 – 3.8 $kg/m^3.d$ in the BF4 system and from 0.6 – 4.6 $kg/m^3.d$ in system BF6 throughout the evaluation period. The volumetric loading of solids, measured as TSS, also varied throughout evaluation of the experimental systems. Total and bed solids loadings were calculated via Equations 3.1 and 3.2, replacing $CBOD_5$ concentrations with TSS concentrations. Bed loadings of TSS ranged from 1.6 to 30.5 in system BF4 and from 2.0 to 80.8 in system BF6. The total volumetric solids loadings applied to the systems ranged from 0.5 to 2.4 and 0.3 to 3.0 for systems BF4 and BF6 respectively.

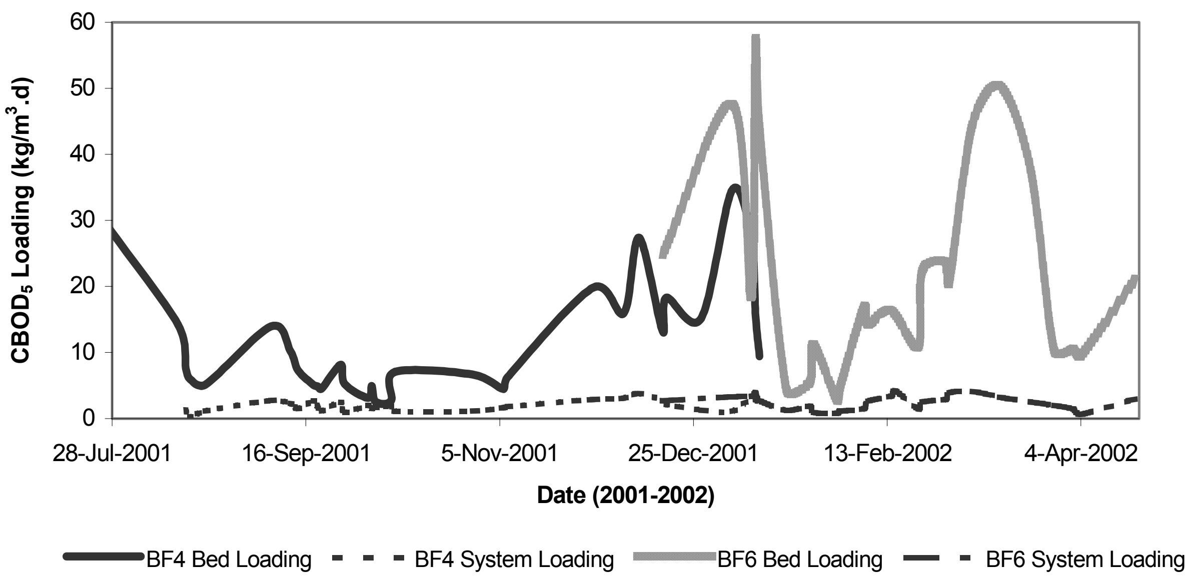


Figure 3-8. Total system organic loadings applied to systems BF4 and BF6 throughout the experimental runs peaked at 3.8 and 4.3 kg/m^3.d, respectively. Bed loadings peaked at 34.6 for BF4 and 56.9 kg/m^3.d for BF6.

PHYSICAL AND BIOLOGICAL TREATMENT IN SLDM FILTERS

Solids and organic materials are simultaneously removed in SLDM filters, as exhibited in Tables 3-2 and 3-3. Results from the two different filter configurations evaluated in this study were divided into different data sets based on the organic conditions inside of the filter bed. This was determined by averaging the concentration of $CBOD_5$ in the fluid immediately prior to and immediately following the bed of filter media. The resulting categories, referred to as low, middle, and high organic conditions, represent different planes of desired filter operation and it is suspected they also represent different locations on the Monod kinetic curve. The low organic substrate regime in this study describes the condition in which the average concentration of $CBOD_5$ in the bed of media was equal to or less than 10 mg/L. The middle and high level substrate range indicates the average filter bed $CBOD_5$ concentration was between 10 and 30 mg/L and above 30 mg/L, respectively. It should be noted that the data presented in Table 3-2 reflects multiple passes through the filter bed, each pass lasting from 30 to 90 seconds.

Table 3-2. Mean filter results under different organic substrate conditions within the filter bed.

Experimental Series	$CBOD_5$			TSS		
	Total Loading ($kg/m^3.d$)	Effluent (mg/L)	Removal (%)	Total Loading ($kg/m^3.d$)	Effluent (mg/L)	Removal (%)
BF4 – Low Level (n)	1.5 ± 0.6 (18)	4.43 ± 1.3 (18)	95.4 (18)	0.7 ± 0.2 (7)	3.3 ± 1.7 (7)	93.7 (7)
BF4 – Mid Level (n)	2.5 ± 0.8 (11)	14.2 ± 3.9 (11)	88.2 (11)	1.6 ± 0.6 (8)	14.3 ± 4.5 (8)	80.1 (8)
BF6 – Low Level (n)	1.2 ± 0.4 (4)	4.1 ± 1.9 (3)	96.9 (4)	0.6 ± 0.3 (3)	6.7 ± 7.8 (3)	90.2 (3)
BF6 – Mid Level (n)	2.0 ± 1.0 (10)	14.1 ± 3.7 (10)	88.6 (10)	1.3 ± 0.7 (10)	22.1 ± 7.5 (10)	69.4 (10)
BF6 – High Level (n)	3.3 ± 0.7 (10)	49.8 ± 17 (5)	69.5 (5)	2.1 ± 0.7 (8)	52.0 ± 26 (8)	51.6 (8)

* (n) indicates the number of sampling events
**The standard deviation follows the ± sign

The total organic and the total solids loading rates presented in Table 3-2 were based on the daily flow to the system, and were calculated via Equation 3.1. The average effluent concentrations presented represent the effluent from the system, after multiple passes. Likewise, the percent removal represents the efficiency of the SLDM filters based on the total residence time. This loading rate should be differentiated from the bed loading, the load applied to the filter, and the single pass removal percentages, which are listed in Table 3-3. Values listed in

Table 3-3 were calculated using Equation 3.2 and are based on a "snapshot" of single pass behavior in a recirculation mode.

Table 3-3. Single pass results for the different organic substrate conditions within the filter bed.

	$CBOD_5$		TSS	
	Bed Loading ($kg/m^3.d$)	Single Pass Removal %	Bed Loading ($kg/m^3.d$)	Single Pass Removal %
BF4 – Low Level	6.4 ± 2..9	21.1	4.2 ± 1.5	12.7
(n)	(16)	(12)	(7)	(6)
BF4 – Mid Level	17.6 ± 7.1	11.9	18.4 ± 7.6	19.6
(n)	(12)	(11)	(8)	(8)
BF6 – Low Level	5.3 ± 0.2	29.5	8.5 ± 5.6	22.9
(n)	(2)	(2)	(3)	(3)
BF6 – Mid Level	14.8 ± 5.6	18.7	23.4 ± 7.3	18.7
(n)	(9)	(9)	(10)	(10)
BF6 – High Level	37.7 ± 13.6	14.4	35.7 ± 20.6	18.7
(n)	(5)	(4)	(8)	(7)

* (n) indicates the number of sampling events
**The standard deviation follows the ± sign

Organic Loading

The performance data obtained from both experimental systems BF4 and BF6 were used to evaluate the relationship between total $CBOD_5$ volumetric organic loading and effluent quality. Figure 3-9 illustrates this performance. Using this graph, the effluent concentrations resulting from a corresponding applied load may be determined. In the BF4 unit, applied loads of 2.7 and 4 $kg/m^3.d$ of $CBOD_5$ should result in effluent concentrations of 10 and 20 mg/L, respectively. The upward shift of the trendline for BF6 indicates a performance limitation relative to BF4. This may have been caused by a backwashing complication specific to the unit design.

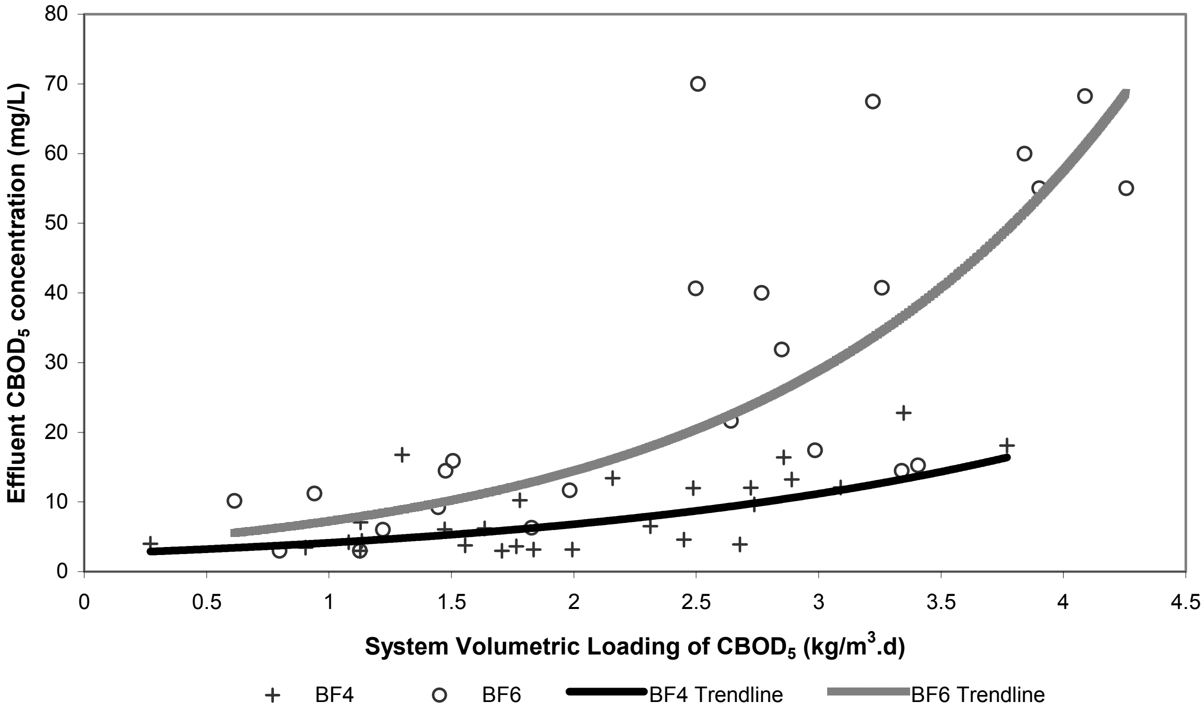


Figure 3-9. Organic loading results revealed a higher loading capacity for the BF4 system when compared to BF6. Loadings up to 2.7 kg/m^3.d resulted in effluent $CBOD_5$ concentrations of 10 mg/L for the system.

The BF4 and BF6 units were backwashed from once every six hours to as frequently as once per 45 minutes in this study. Backwashing frequency exhibited some control over the TSS concentration in the system effluent. Immediately after a backwashing, higher levels of TSS were observed, presumably from abraded biofilm and loosened particulates. The BF4 and BF6 units were designed to prevent discharge immediately after backwashing by hydraulically directing the solids generated by the backwash into the charge chamber. Once in the charge chamber, the solids were expected to settle and the supernatant was allowed to slowly reenter the filter bed. Since BF4 recirculated on an external tank this strategy worked well; however, in the concentrically designed BF6 unit openings in the sludge storage area allowed the turbulent conditions generated during backwashing to cause a “dirty backwashing”. This condition was further aggravated by the highly variable flow at the site causing large slugs of influent to almost immediately cause discharge after a backwashing event. On one such occasion the TSS concentration of a sample taken soon after overflow resumed following a backwash was found to be twice as high (at 44.2 mg/L) than a sample taken thirty minutes later (21.4 mg/L). Other researchers have referred to this phenomenon as a "suspended solids overshoot" (Kim and Yoo, 2001).

Water temperatures in southern US states range from 22-36°C in the summer, 6-16°C in the winter, and 12-25°C during other seasons (Wu, 2003). By allowing the experimental units to operate over a wide range of ambient south Louisiana temperatures, the influence of temperature on $CBOD_5$ removal was studied. Results show the removal of organic matter is not sensitive to temperature during summer conditions, namely at temperatures greater than 21°C, as shown in Figure 3-10. Below temperatures of 21°C and at applied organic bed loading rates greater than 14 kg/m^3.d, unit BF4 exhibited more sensitivity to temperature. As opposed to the BF4 system,

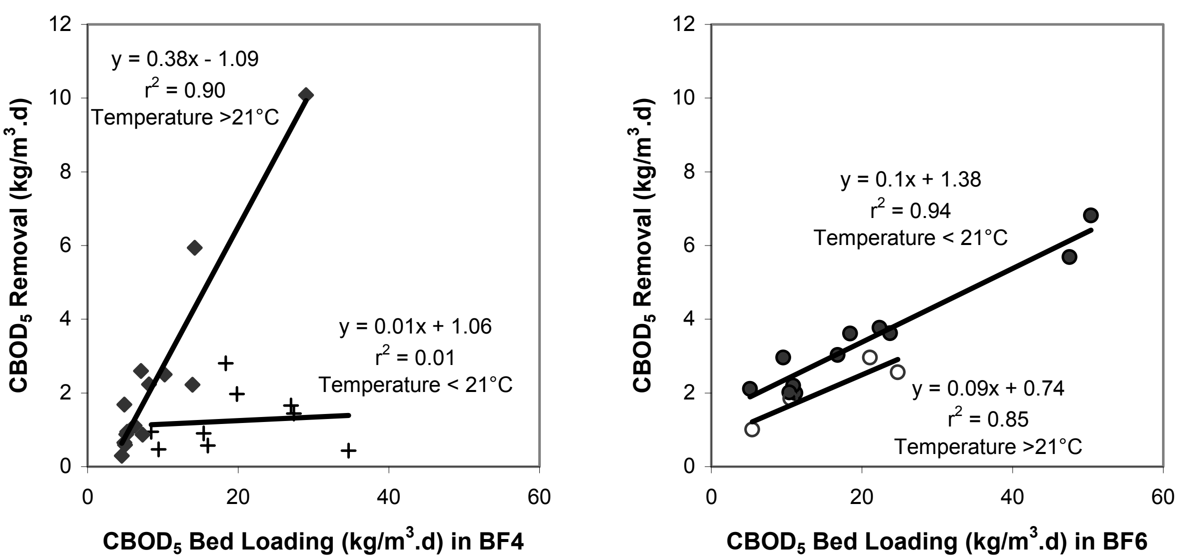


Figure 3-10. A comparison of removed BOD loads to applied BOD loads indicated no sensitivity to temperatures greater than 21°C in BF4.

there was no statistically significant difference exhibited in the BF6 unit, although this may be attributed to complications from dirty backwashing.

The SLDM filters evaluated were capable of removing up to 10 kg/m^3.d of $CBOD_5$, based on single pass data. $CBOD_5$ removal ($CBOD_r$) was calculated via the following equation:

$$CBOD_r = \frac{(CBOD_{in} - CBOD_{out})Q_r}{V_b} \qquad \text{Equation 3.3}$$

Where $CBOD_{in}$ represents the influent concentration of $CBOD_5$ to the biofilter system, in mg/L; $CBOD_{out}$ is the effluent concentration of $CBOD_5$ from the biofilter system, in mg/L; and Q_r in m^3/d and V_b in L, as previously defined.

<u>Solids Removal</u>

System BF4 facilitated an average TSS removal of 84% (n=19), while BF6 experienced an average TSS removal of 70%. Figure 3.11 illustrates the relative performance of the experimental units in terms of solids removal. Again a limitation in performance is seen is BF6, most likely a result of the dirty washing phenomenon.

The removal of suspended solids was an important consideration in these units when aiming to reduce total $CBOD_5$ concentrations for single and multi-pass regimes. The fraction of particulate bound $CBOD_5$ in the wastewater was intermittently tested during filter evaluation. The fraction was found to increase from 0.37 to 0.73 as the wastewater traveled from the influent point until it exited from the filter. Soluble $CBOD_5$ was consistently found to be less than 5 mg/L. Likewise, the VSS to TSS ratio in both filters increased as the wastewater traveled through the treatment train. The TSS was approximately 90% volatile in the system influent, increasing to nearly 100% to 92% volatile in the effluents of BF4 and BF6, respectively. The

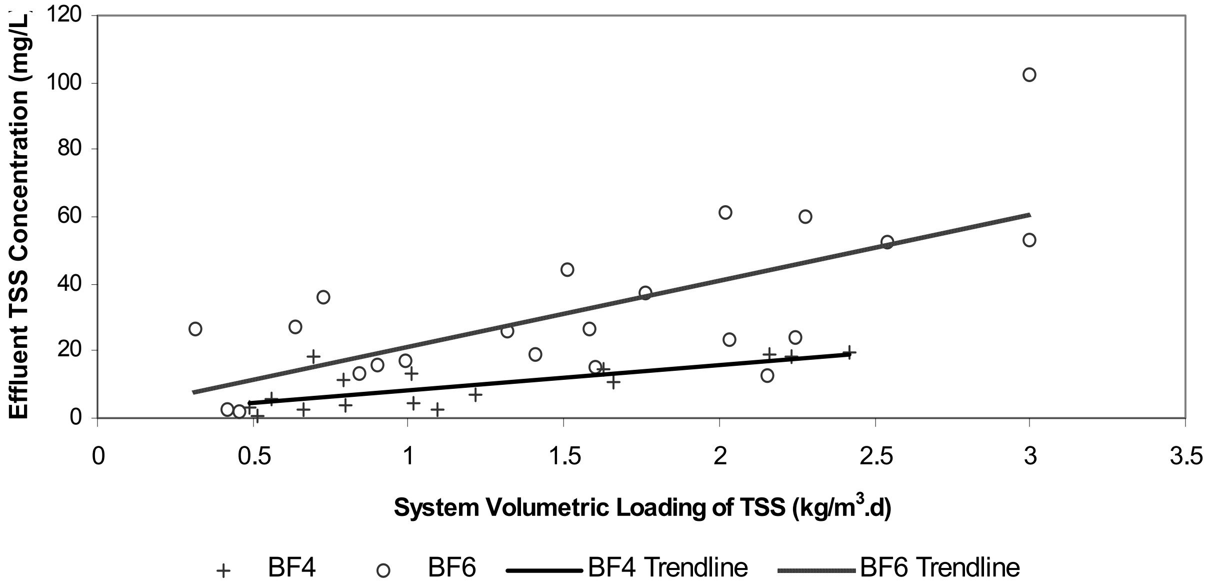


Figure 3-11. Applied solids loading curves show a relatively better performance of BF4. Solids loadings up to 1.3 kg/m^3.d resulted in a system effluent TSS concentration of 10 mg/L.

relationship between TSS and $CBOD_5$ in the effluent from the system reveals that a higher TSS removal would lower $CBOD_5$ concentrations. Positive correlations were found for both filters; Figure 3-12 displays the relationship in BF4.

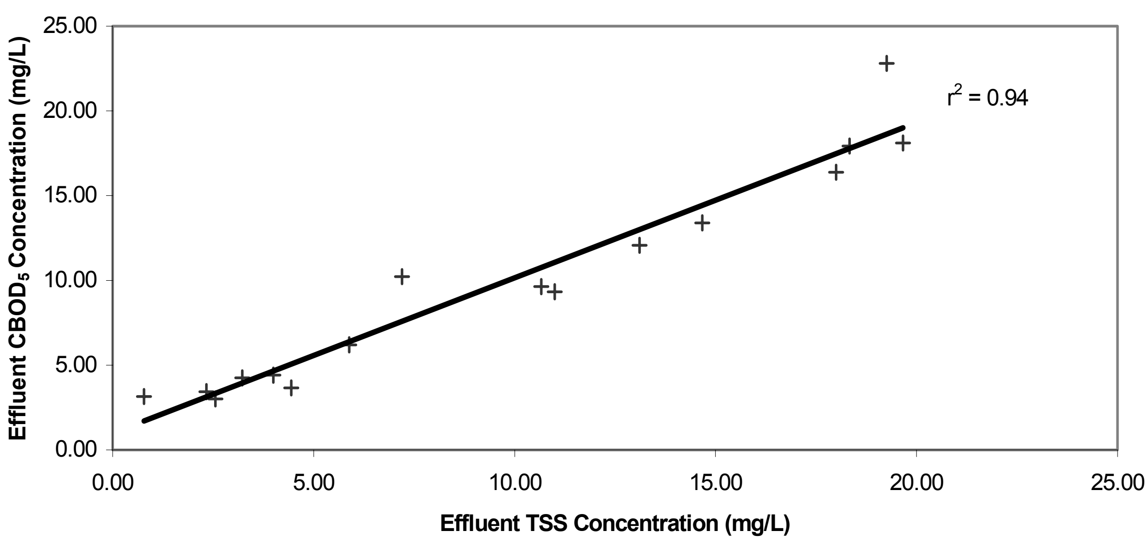


Figure 3-12. The effluent TSS and $CBOD_5$ concentrations were well correlated, indicating organic levels could be reduced by increased solids removal.

OPERATIONAL PARAMETERS

Operational parameters such as retention time, filtration rate, and oxygen availability can greatly affect the performance of both the biological and physical functions of SLDM filters. Some of the operational parameters for the same experimental regimes described previously can be found in Table 3-4.

Table 3-4. Operational parameters for different organic substrate conditions within the filter bed.

Experimental Series	Flow Ratio Q_r/Q	Filtration Rate (m/h)	Retention Time (min) One Pass	Retention Time (min) Total	Oxygen Uptake Rate ($kg/m^3.d$)
BF4 – Low Level (n)	71.3 ± 28 (18)	13.9 ± 2.8 (18)	0.76 ± 0.2 (18)	51.6 ± 17 (18)	2.5 ± 0.6 (18)
BF4 – Mid Level (n)	47.4 ± 13 (11)	13.0 ± 3.9 (12)	0.87 ± 0.4 (12)	39.4 ± 15 (11)	1.6 ± 0.6 (12)
BF6 – Low Level (n)	104.8 ± 41 (4)	13.2 ± 1.3 (4)	1.0 ± 0.1 (4)	98.3 ± 30 (4)	1.6 ± 0.2 (4)
BF6 – Mid Level (n)	61.3 ± 32 (10)	13.1 ± 3.8 (10)	1.0 ±0.3 (10)	62.4 ± 33 (10)	1.3 ± 0.5 (10)
BF6 – High Level* (n)	32.9 ± 10 (10)	10.8 ± 4.2 (10)	1.3 ± 0.5 (10)	40.0 ± 9 (10)	0.6 ± 0.4 (10)

*High Level bed concentration samples were consistently oxygen limited. DO concentrations in the filter bed averaged less than 1 mg/L.

Evaluation of SLDM filters must not be done on the basis of system loadings alone. The bed loadings, driven by changes in the recirculation flow rate, should also be considered when assessing performance of these filters. Changes in the recirculation flow rate impact several important operational parameters. A decrease in Q_r, for example, would proportionally decrease the filtration rate and increase the one pass retention time. As the one pass retention time increases, the oxygen uptake rate (OUR) would increase, provided the conditions in the filter were oxygen limiting, which is much more likely than encountering a substrate limitation in

SLDM filters. Consequently, the removal of BOD (BOD_r) would increase with an increase in OUR. This would cause a downward shift in the loading curve, for both the total system and the bed loading curves. Since SLDM filters incorporate recirculation of the wastewater through their beds several times as a means to provide sufficient external aeration, the flow rate through the filter is larger than the daily flow rate. For the BF4 and BF6 systems, the recirculation flow rate, Q_r, was on average 60 to 45 times larger than the daily flow rate of the system, Q.

The filtration rates of 10 to 15 m/h in this study were similar to rates found in other floating media filters (Ødegaard and Helness, 1999; Sampa and Tanaka, 1995). Higher solids removal rates have been associated with lower filtration rates in floating medium filters (Ødegaard and Helness, 1999; Tanaka *et al.*, 1995). The filtration rate was calculated via Equation 3.4.

$$\text{Filtration Rate} = \frac{Q_r}{\left(V_b / h\right) 24} \qquad \text{Equation 3.4}$$

Where Q_r is the recirculation flow rate in m^3/d, V_b is the media volume in m^3, h is the height of media in the filter bed in m, and 24 is used to convert days to hours.

Total retention times in the filter ranged from 40 to 100 minutes. Higher retention times were associated with lower bed concentrations. While the influence of retention time was not isolated in this study, a previous study found that residence time had only little influence, unless particle hydrolysis occurred (Ødegaard *et al.*, 2000). Particle hydrolysis was found by Ødegaard *et al.* (2000) to occur after 2-3 hours. The equations used to calculate one pass and total residence times follow:

$$\text{One Pass Retention Time} = \frac{V_b * \varepsilon * 1440}{Q_r} \qquad \text{Equation 3.5}$$

$$\text{Total Retention Time} = \frac{V_b * \varepsilon * 1440}{Q} \qquad \text{Equation 3.6}$$

Where V_b in both equations is the Volume of filter media in m^3, ε is the porosity, and 1440 is used to convert from units of days to units of minutes. In the one pass residence time equation, Q_r is the recirculation flow rate in m^3/d, whereas the term Q used in the total residence time equation is the total flow rate applied to the entire system, in units of m^3/d.

The SLDM filters used in this study supplied oxygen through recirculation of the wastewater through the filter bed by means of airlift pumps. The wastewater made multiple passes through the filter bed with retention times of 30 seconds to one and a half minutes per pass. At the end of each pass, an airlift pump returned the wastewater to the equalization basin chamber. The airlift pump served the dual functions of water delivery and aeration.

The relationship between dissolved oxygen concentrations in the filter and $CBOD_5$ reduction was used to evaluate filter performance. Although a complex function of microbial population, loading to the filter, solids removal efficiency, and operational parameters, a generic, simplified relationship was used to evaluate overall performance at different bed concentrations. The strategy was to relate the dissolved oxygen uptake rate (OUR) of the bacteria to the removal rate of $CBOD_5$ (BOD_r) applied to the filter by a function labeled the MX Factor. The oxygen uptake rate (OUR) was calculated via Equation 3.7.

$$OUR = \frac{(DO_{in} - DO_{out})Q_r}{V_b} \qquad \text{Equation 3.7}$$

Where DO_{in} and DO_{out} are the concentrations of DO entering and exiting the filter, respectively, and Q_r and V_b as previously defined (Malone and Beecher, 2000). The relationship between BOD_r and OUR is expressed:

$$MX = \frac{BOD_r}{OUR} \qquad \text{Equation 3.8}$$

As shown in Figure 3-13, under periods of very low bed concentration (less than 8 mg/L $CBOD_5$), MX values were consistently less than one. Hu *et al*. (1994) investigated the oxygen uptake characteristics of a submerged biofilter with different sized media. Their study found that when BOD removal was low, endogenous respiration accounted for a large part of the OUR. Ratios below one may have also indicated that some level of nitrification was occurring.

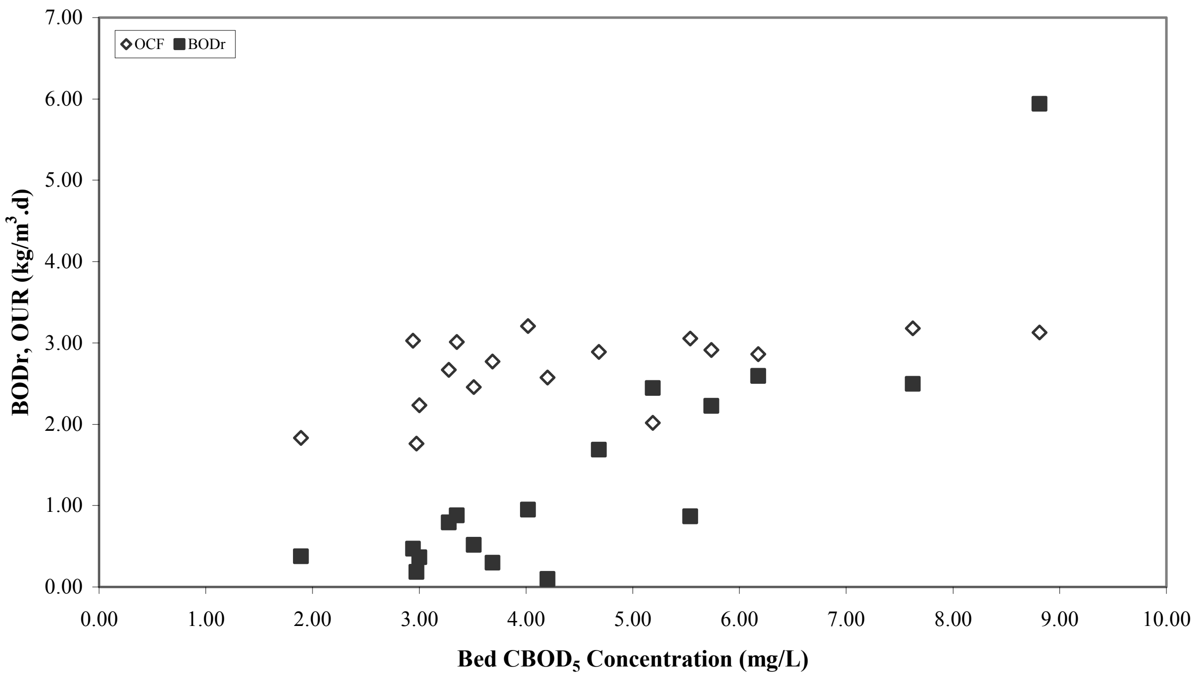


Figure 3-13. At low organic bed concentrations, OUR was smaller than BODr, indicating the occurrence of nitrification.

Conclusions

The consolidation of biological processes and physical operations into a single unit represents the ability of a bioclarifier to function as an entire secondary treatment train and, if protected from the natural oils and greases found in domestic wastewater, as a primary and secondary treatment system in a single unit. Effluent $CBOD_5$ concentrations of 10 mg/L were achievable at organic loadings of 2.7 $kg/m^3.d$ for the BF4 system and 1.5 $kg/m^3.d$ for the BF6 system. Corresponding one pass bed loadings are 15 and 10 $kg/m^3.d$ of $CBOD_5$ for the BF4 and BF6 systems, respectively. Likewise an effluent TSS concentration of 10 mg/L was obtained at TSS loadings of 1.3 and 0.4 for the BF4 and BF6 systems, with corresponding bed loadings of 12 and 9 $kg/m^3.d$.

In addition, the SLDM filter required very little human interaction for operation and requires little maintenance. Results have shown that SLDM filters could potentially applied to domestic wastewater treatment for secondary wastewater treatment. If used for this application, a single SLDM filter could be used to replace an activated sludge unit and secondary clarifier or a trickling filter and its associated secondary clarifier, substantially reducing the construction costs and the land requirement of secondary treatment. A SLDM filter could also be used as an on-site system in areas where traditional septic systems could not be implemented.

The continuing research effort will analyze the performance of different filter configurations under higher levels of organic loading and under different operational parameters, such as backwash frequency, total retention time, and filtration rate.

CHAPTER 4 : MANAGEMENT STRATEGIES FOR STATIC LOW DENSITY MEDIA FILTERS OPERATING AS BIOCLARIFIERS FOR THE TREATMENT OF VARIABLE FLOW DOMESTIC WASTEWATER

Introduction

An effective means for the removal of organic material from domestic wastewater is the addition of a bioclarifier downstream of primary treatment unit operations. Bioclarifiers, which concurrently remove biological and physical contaminants from a wastewater, may be used to replace the classical two-step process of secondary wastewater treatment; namely, biological treatment followed by secondary clarification. Replacement of two units with a single process follows the consolidation strategy, which promotes the reduction of the total number of units and processes in a treatment train by combining multiple functionalities into single structures. Consolidated units must combine the attributes of simplicity, efficiency, and flexibility to successfully replace multiple units. Savings in capital and operating costs along with a reduction in the required footprint for single components serving multiple duties may overcome the disadvantages of decreased individual process optimization and efficiency (Loyless and Malone, 1998).

Static Low Density Media (SLDM) filters have recently been used as bioclarifiers in the treatment of domestic wastewater. These filters, also known as Floating Bead Filters (FBF) or Floating Bead Bioclarifiers (FBBs), have been used to treat aquacultural wastewaters for the past twenty years. SLDM filters are a class of submerged bioclarifiers that contain floating granular medium that not only acts as a biofilm carrier, but also concurrently treats the wastewater physically, by capturing suspended solids in the waste stream via surficial straining, deep bed filtration, and adsorption (Malone and Beecher, 2000). SLDM filters are periodically backwashed via mechanical, hydraulic, or pneumatic means to remove accumulated solids and excess biofilm from the media bed. Figure 4-1 exhibits an example of a pneumatically washed SLDM filter.

Figure 4-1. A variety of operational strategies may be used to influence the performance of Static Low Density Media filters.

While conceptually similar to Biological Aerated Filters (BAFs), the defining characteristic of the static nature of the media bed in SLDM filters is the differentiating point. To maintain static and aerobic conditions within the granular medium filter bed, SLDM filters employ recirculation and external aeration, in contrast to internally aerated BAFs. Whereas both technologies may function either as a complete secondary treatment train for both organic and solids removal, or as a tertiary treatment step, removing both ammonia and solids, the management strategies used to optimize performance differ between the two filters. In SLDM filters, differences in management strategy may provide additional flexibility for potential uses of the filter, adding the abilities to function as: a physical solids separation filter, a contact flocculator, or as a denitrification unit.

The performance of SLDM filters is greatly impacted by a variety of parameters that may be controlled by changes in daily management technique and in filter configuration. The recirculation flow fraction, backwashing frequency, and applied loading are parameters that may be utilized to alter the residence time, oxygen uptake rate, and solids capture abilities. Furthermore, the backwashing mechanism and selection of media can affect the performance of both biological and physical functions of SLDM filters. This paper describes the impacts of several of these parameters and the management strategies that may be used to optimize filter performance.

Methods and Materials

The performance of SLDM filters receiving a variable quantity and quality flow of domestic wastewater was evaluated when subjected to changes in various operational parameters. Batch studies were performed to partially reveal the impact of retention time and backwashing on bioclarifier operation. Two additional experimental units receiving wastewater from an industrial facility in Denham Springs, Louisiana, were evaluated in 2001 and 2002 for their abilities to function as a bioclarifier. The biofilm carrier throughout the entire research effort was enhanced nitrification (EN) modified media, 3 to 5 mm in diameter with a density of 0.90 kg/L, a clean bed porosity of 0.55, and with a total specific surface area of approximately 1100 to 1250 m^2/m^3 (Malone *et al*., 1993). The SLDM filters in this study were operated at filtration rates between 5 and 30 m/h, classified as high filtration rates by Ødegaard and Helness (1999). Backwashing frequencies were set from once every 0.75 hours to once every 8 hours.

Periodically, a few gallons of sludge were discharged from the filters, typically once per week. Descriptions of the site characteristics and the experimental setup and methodologies of each experimental system may be found in Appendix A.

SITE CHARACTERISTICS

The experimental units were in operation outside of a small industrial facility in Denham Springs, Louisiana, USA. The wastewater was largely domestic in nature, and was generated only when the employees were working at the facility. The site was therefore subject to highly variable flow characterized by morning and afternoon peaks, no overnight flow, and no weekend flow. Ambient outdoor temperatures caused the temperature of the wastewater to vary from 7 to 32°C during the evaluation period. Raw wastewater from the facility was intermittently pumped from a below ground sump into a 3.79 m^3 (1000 gallon) fiberglass tank that functioned as both a storage tank and also as a primary clarifier, however not as a flow equalization basin. Influent to the experimental systems was gravity fed directly from this tank. Flow delivery to the experimental units was proportional to the rate of wastewater generation by the industrial facility. Since the units were fed with real wastewaters, uniform BOD and TSS concentrations were not achieved in all phases of the study. The average influent wastewater characteristics for the experimental units are given in Table 4-1.

Table 4-1. Influent Wastewater Characteristics

	Batch (time 0)	BF4	BF6
$CBOD_5$, mg/L	47	104	145
(n)	(3)	(33)	(25)
TSS, mg/L	37	60	91
(n)	(3)	(20)	(22)
Temperature C	32.2	22.6	17.7
(n)	(3)	(37)	(25)

BATCH STUDIES

Three batch studies were performed on June 1, June 13, and June 20 of 2001 on an experimental system, BF3. During normal operation, the BF3 system was comprised of a SLDM filter with internal recirculation of the wastewater via and internal airlift pump. The hull of experimental system BF3 was constructed of aluminum. The filter contained 14.2L (0.5 ft^3) of EN media biofilm carrier. Below the filter bed was a backwash air chamber outfitted with a trigger, similar to those in experimental units BF4 and BF6. Experimental pilot testing on the BF3 system was executed in the spring of 2001, however due to the inability of the system to dampen shock loadings at the testing facility, traditional testing was abandoned and batch tests were performed. For the batch studies, a fiberglass tank was filled with approximately 250 liters of wastewater and a closed loop was made with the SLDM filter. The wastewater was allowed to recirculate and hourly samples were taken for a period of five hours.

Figure 4-2. Batch studies were performed on the BF3 filter, which was a modular, internally recirculating SLDM design.

ANALYTICAL METHODS

Temperature, pH, and flow measurements were recorded along with other operational parameters, such as backwash frequency, during each sampling event. Water quality parameters were tested in triplicate according to Standard Methods and include the following: $CBOD_5$ (5210B), DO (4500-O C), TSS (2540 D), and VSS (2540 E) (APHA, 1995). Dissolved oxygen samples were preserved immediately after collection and were brought to the Water Quality Laboratory at LSU, along with the other samples, for analysis.

Backwashing

As submerged granular medium biofilters operate, the captured suspended solids and biofilm growth contribute to headloss buildup within the filter. Periodic backwashing of the filter is required to prevent clogging of the media bed. This practice results in a semi-continuous operation of the biofilter, with filtration cycles interrupted by backwashes (Yoo and Kim, 2001). Halting of treatment has been cited as one of the disadvantages of submerged granular medium filters (Ødegaard *et al*., 1994). Alternate technologies have been developed for use in SLDM filters in an effort to avoid the drawbacks of substantial water loss, large energy input requirements, and interruption of treatment associated with backwashing. These include: mechanical washing via propellers, hydraulic washing, and pneumatic washing, along with use of floating media as the biofilm carrier. While all of these backwashing mechanisms have filtration cycles and backwashing cycles, backwashing in SLDM filters can occur in less than one minute and does not interrupt flow application to the filters.

The backwashing classification was the same for all of experimental units tested in this study (BF3, BF4, and BF6), namely, pneumatic. In this particular type of SLDM filter, air is introduced into an airtight “charge” chamber beneath the bed of granular medium, at a rate set by the operator. Once the charge chamber has reached its volumetric capacity of air, a pneumatic trigger fires and the backwashing mode commences. During backwashing, which typically lasts for less than twenty seconds, air from the charge chamber is released into the filter bed, agitating the media. This action concurrently shears accumulated solids and excess biofilm from the media and interstitials of the bed. During backwashing, neither air nor water application to the filter is interrupted. As the air leaves the charge chamber and is introduced into the filter, the water level in the filter drops below the discharge level, thus effluent is not released from the filter. Once the charge chamber ceases to release air, the static bed reforms and floats upwards, and the filter reenters its filtration mode. The water that dropped from the bed enters the charge chamber, which serves a dual function as a sludge settling and storage area. As the solids in the backwash waters settle, the supernatant is again passed through the filter bed, while the charge chamber is recharged with air. Sludge may be drained manually or via control systems from the sludge outlet in periodic intervals set by the operator.

Management of backwashing has been found to be a critical consideration in addition to proper design for maintaining desired effluent qualities in floating granular medium filters (Yoo and Kim, 2001). Inefficient or inappropriate backwashing of may lead to many problems including poor filter performance (Fitzpatrick, 1998). While backwashing is typically the means for reducing headloss, the quality of the backwash, influenced by such factors as mechanism, degree of abrasion, intensity, and frequency, may be used to control biofilm if water loss is kept to a minimum. Biofilm control, including the management of biomass concentration, biofilm thickness, and biofilm morphology, is necessary for the stable operation of bioreactors (Tijhuis, 1996).

Biofilm thickness and morphology, in particular, have been found to be greatly affected by abrasion intensity; as applied detachment forces increase, stronger biofilms develop (Tijhuis, 1996). Increased shearing forces, or increased backwash intensity, are typically accompanied by an increased degree of abrasion, and therefore an increased fraction of solids removed by the backwash. In BAF systems, which are frequently hydraulically backwashed, high intensity backwashes are attributed to high hydraulic loads of backwash water. High intensity backwashes have been found to excessively shear biofilm from the BAF media, creating an undesirable condition in which lengthy time periods are required to recover sufficient biofilm structure (Yoo and Kim, 2001).

The strength of the shearing forces applied during backwashing of SLDM filters is dependent on the backwashing technique employed; typically mechanical (propeller) washed filters apply the most intense wash, while pneumatic washing provides a more gentle wash. Figure 4-3 illustrates the impact of backwash intensity over a wide range of backwash intervals on the nitrification capacity of SLDM filters on an areal basis. Intensity of the shear forces applied to the microbial consortia can impact biofilm morphology, growth rate, and metabolism (Liu and Tay, 2001). Preferential growth within the microbial film may be generated by manipulating the frequency and intensity of backwashing. In tertiary treatment applications, a frequent and gentle backwashing regime can

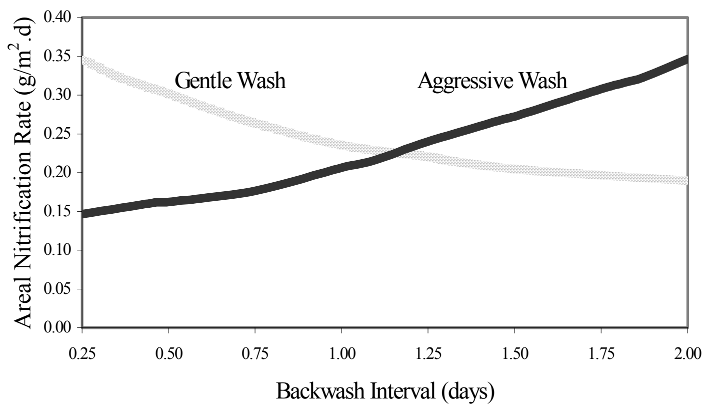


Figure 4-3. Nitrification rates in SLDM filters treating aquacultural wastewaters have been found to be dependant on both backwash intensity and frequency (after Golz, 1997).

successfully remove solids and heterotrophic bacteria while maintaining a nitrifying biomass in granular medium biofilters (Boller *et al*., 1997; Golz, 1997; Ohashi *et al*., 1995). However, overly frequent and aggressive (high shear stress) washing can hinder nitrification by removing the requisite nitrifying bacteria (Golz *et al*., 1999; Golz, 1997; Malone *et al*., 1993).

Along with biofilm management, selection of backwashing frequency can impact several other facets of biofiltration performance. Solids accumulation within a SLDM filter is controlled by the backwashing regime. Removal rate of solids in SLDM filters is determined by the frequency of backwashing and the fraction of solids removed with each wash (Golz, 1997; Sastry, 1996). Frequent backwashing minimizes headloss, thereby maximizing fluid throughput, which is critical for oxygen transfer. The external aeration strategy used in aerobic SLDM filters benefits from bed depths, of 61 cm (24 in) and less, to fully utilize the dissolved oxygen present in the water for aerobic biological processes. A slowdown of the recirculation flow, which could result in potentially undesirable anaerobic conditions towards the effluent side of the filter, should be avoided in aerobic SLDM fitlers. In addition to headloss control, frequent backwashing also reduces the sludge retention time within the media bed, which reduces the biochemical load on the system and focuses the biological activity within the filter on SBOD removal. Particle hydrolysis is known to occur within 2 to 3 hours (Ødegaard *et al*., 2000). The backwash frequency requirement in SLDM filters is affected by influent TSS concentration, hydraulic application rate, and media properties related to solid accumulation capacity. To date, SLDM filters operating as bioclarifiers in domestic wastewater treatment applications have been backwashed at highly frequent intervals of once per half hour to once every eight hours.

While frequent backwashing has several benefits, one disadvantage, which can be compensated for by proper design and maintenance of bioclarifiers, remains. Immediately following a backwash in BAFs, a sharp increase in suspended solids concentration in the filter effluent, referred to as the "SS Overshoot" has been observed. (Moore *et al.*, 2001; Yoo and Kim, 2001). This phenomenon has also been witnessed in SLDM filters and is also referred to as "dirty backwashing". In batch study 3, shown in Figure 4-4, the SS Overshoot increased TSS concentrations by 25% following a backwashing at 2.5 hours. Filtration rates for batch study 3 increased from 9.0 to 13.8 m/h throughout the duration of the study.

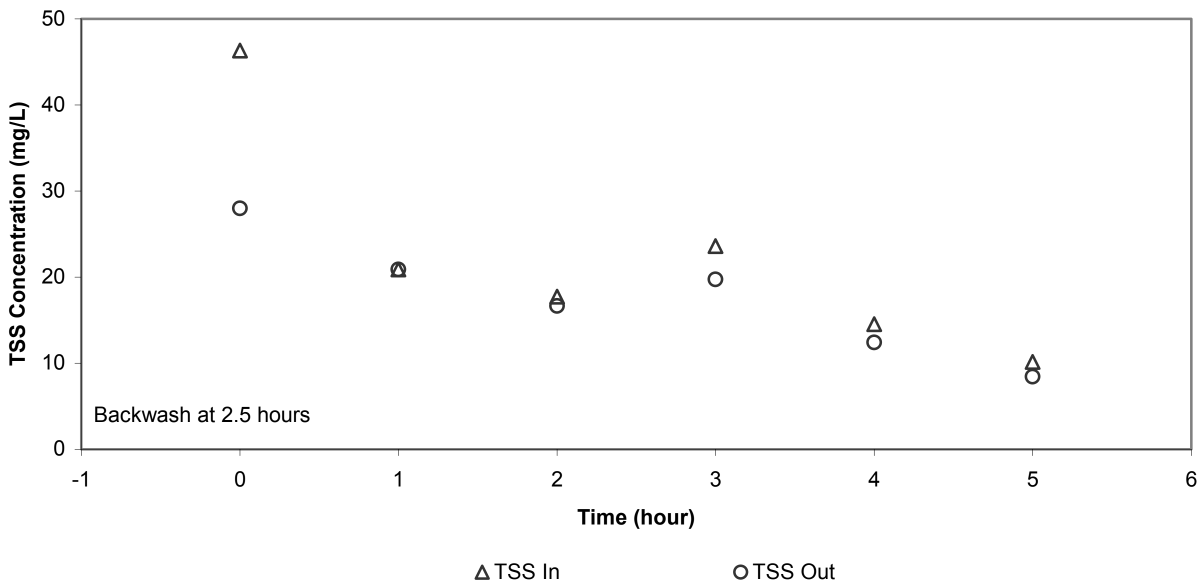


Figure 4-4. SS Overshoot was observed during all of the batch studies. In batch study 3, the average bed TSS concentration 30 minutes after backwashing was 25% higher than values from 30 minutes prior to backwashing.

SS Overshoot remained problematic in experimental systems BF4 and BF6. Figure 4-5 illustrates the relationship between effluent TSS and the frequency of backwashing, given in intervals of hours, in the BF4 system. Within the backwash interval range of 0.75 to 2 hours, TSS and COD_5 concentrations were found to be elevated at the shortest intervals due to backwash effects. Beyond the initial overshoot of suspended solids, TSS concentrations decrease as time between backwashes increases, although after some time the capacity for the filtration bed to hold additional solids will decrease, as headloss buildup in the media bed results in reduced throughput. Recovery from SS overshoot in BAFs was found to occur more quickly when lower filtration rates were applied to the filter following the backwash (Moore *et al.*, 2001).

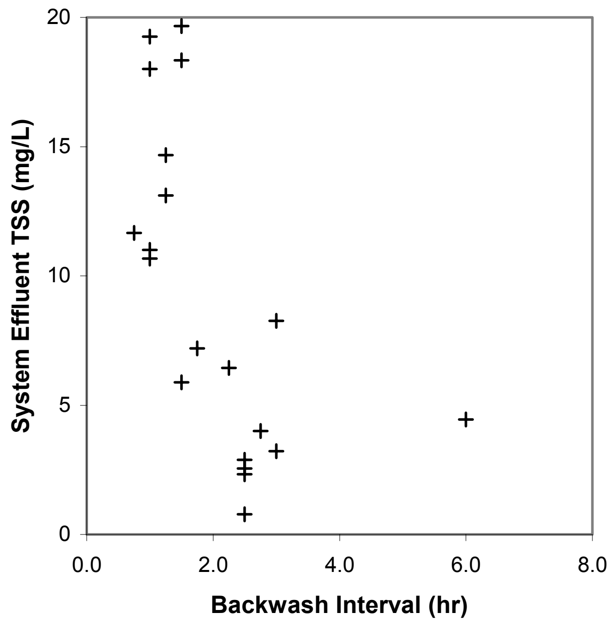


Figure 4-5. Dirty backwashing caused inflated effluent TSS concentrations at short backwash intervals.

Proper design of SLDM filters may limit the dirty backwashing, or SS overshoot, effect. The BF4 and BF6 units were designed to prevent discharge immediately after backwashing by hydraulically directing the solids generated by the backwash into the charge chamber. Once in the charge chamber, the solids were expected to settle and the supernatant was allowed to slowly reenter the filter bed. Since BF4 recirculated on an external tank this strategy worked relatively well, although design modifications may have reduced backwashing impact. In the concentrically designed BF6 unit, openings in the sludge storage area allowed the turbulent conditions generated during backwashing to cause a dirty backwashing. This condition was further aggravated by the highly variable flow at the site causing large slugs of influent to almost immediately cause discharge after a backwashing event. On one such occasion the TSS concentration of a sample taken soon after overflow resumed after a backwash was found to be twice as high (at 44.2 mg/L) than a sample taken thirty minutes later (21.4 mg/L). Design modifications enclosing the charge chamber, and thus the stored sludge and backwash water, dramatically reduced the SS overshoot in the concentric design (Bellelo *et al.*, 2003). Time between backwashing should not be maximized based on solids capture in aerobic SLDM filters, however. Backwash interval selection in aerobic SLDM filters is typically dominated by the need to maintain aerobic conditions within the filter.

The act of backwashing affects the biochemical functions of SLDM filters, as well as the physical clarification capacity. During backwashing, shearing forces act are applied to the biofilm carriers, removing solids from the interstitials of the media and abrading the biofilm. These forces stress the biofilm consortia, initially causing a reduction in the oxygen uptake rate (OUR) and the biochemical oxygen demand removal (BODr) rate. Data from BF4 prototypes tested indicate the oxygen uptake rate (OUR) in the filter is significantly lower at backwash frequency intervals of less than 1.5 hours. After a recovery period, these rates reach typical operating rates, dependant on loading rates and environmental conditions.

Recirculation Flow

The static nature of the media matrix is a defining characteristic of SLDM Filters. Maintaining a static granular media bed, which should result in improved solids capture abilities, requires recirculation of the flow and external aeration if aerobic conditions are to be maintained in the media bed. For aerobically operated SLDM filters, recirculation flow is a parameter of utmost importance. Changing the recirculation flow rate also impacts other operational parameters, such as residence time and filtration rate. If one were to decrease the Q_r to Q ratio, for example, the filtration rate would proportionally decrease and the one pass retention time would increase. As the one pass retention time increases, the oxygen uptake rate (OUR) would increase, provided the conditions in the filter were oxygen limiting, which is much more likely than encountering a substrate limitation in SLDM filters. Consequently, the removal of BOD (BOD_r) would increase with an increase in OUR. This would cause a downward shift in the loading curve, for both the total system and the bed loading curves. Since SLDM filters incorporate recirculation of the wastewater through their beds several times as a means to provide sufficient external aeration, the flow rate through the filter is larger than the daily flow rate. For the BF4 and BF6 systems, the recirculation flow rate, Q_r, was on average 60 to 45 times larger than the daily flow rate of the system, Q.

RESIDENCE TIME

Experience has shown that fixed film reactors are capable of operating at much lower residence times than suspended growth reactors while providing equivalent substrate removal (Meunier and Williamson, 1981). Hydraulic residence times in biofilm reactors can be an order of magnitude smaller than activated sludge systems (Morgenroth *et al*., 2002). In fixed film reactors, the total time of contact of wastewater to the biofilm inside of a biofilter is critical at short residence times, with importance waning at longer residence times, similar to a Monod

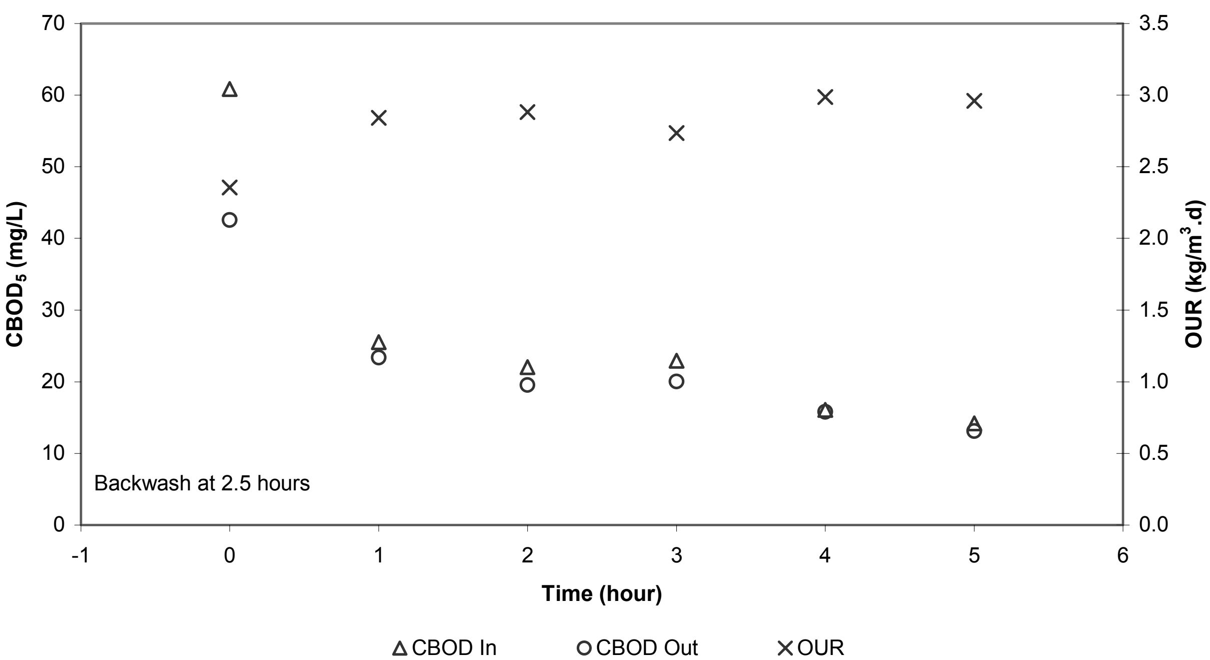


Figure 4-6. Degradation in SLDM filters occurs primarily within the first hour of residence.

kinetics curve. Batch studies were performed to evaluate the importance of residence time in SLDM filters on organic and solids removal. Typical results from the batch studies, as seen in Figure 4-6, indicate the majority of organic degradation within the filter occurred within the first hour, when flow rates of 3 to 1 l/min were applied to 14.2 L of media. Solids removal followed a similar trend in the batch studies, while OUR levels remained relatively constant around 3 kg/m^3.d. Time 0 in Figure 4-6 indicates the time of the first sample taken in the batch study, and not the initial conditions within the reactor.

Performance efficiency of the SLDM filters can be evaluated on a comparative basis to other fixed-film systems based on this and other factors. An 85% removal of $CBOD_5$ was achieved within 40 minutes in the BF4 experimental system, when treating low strength domestic wastewater. By comparison, in a study sited by Kinner and Eighmy (1989), a downflow BAF was able to remove 85 and 95 percent of the organic carbon and TAN, respectively, during a range of residence times from 60 to 90 minutes. Table 4-2 summarizes the ability of different biofilm reactors to remove organic material based on retention time within the biofilter.

Table 4-2. Short residence times are achievable in different biofilm reactors, including SLDM filters.

Reactor Type	Residence Time	Removal	Source
BF4	40 minutes	85% $CBOD_5$	
BF6	60 minutes	85% $CBOD_5$	
MBBR with KMT	30 minutes	72-82% COD	(Liao et al., 2003)
BAF with LDPE	38 minutes	83% COD	(Belgiorno et al., 2003)

LDPE = low density polyethylene

OXYGEN UPTAKE RATE

Oxygen is essential to submerged aerobic biofilters; oxygen-limiting conditions can severely impair a filter to the point of failure. Residual oxygen levels of 0.5 to 2.0 mg/L are required to prevent oxygen limiting conditions (Davis and Cornwell, 1998). The transfer of oxygen in biofilms is affected by fluid velocity, biofilm morphology, biofilm age, and substrate concentration in the bulk liquid (Kinner and Eighmy, 1989; Hickey, 1988). In an effort to supply adequate oxygen to microbial films, various different biofilm reactor configurations have been developed that manipulate various oxygen delivery mechanisms. In recent years, BAFs have received attention due to their capacity to supply oxygen while submerged, by injecting diffused air into the bottom of the reactor. Externally aerated SLDM filters lend a variety of possible aeration techniques, such as using diffusers in a recirculation sump or passing the liquid through a contact aerator. Airlift pumps, also used in SLDM Filters, have been shown to provide sufficient oxygen for recirculating aquatic systems, although they are admittedly less energy efficient (Loyless and Malone, 1998). The efficiency discrepancy can be attributed to the water circulation element that airlifts provide simultaneously with aeration.

The Oxygen Uptake Rate (OUR) has been used to evaluate SLDM filters functioning as tertiary treatment for aquacultural waste. OUR, also known as oxygen consumed in the filter (OCF) (Manthe *et al*., 1988) in the aquaculture arena, was successfully used by Malone and Beecher (2000) in conjunction with a volumetric total ammonia nitrogen (TAN) conversion term to evaluate the performance of nitrifying biofilters, specifically the fractions of oxygen used for nitrification and organic oxidation were calculated. The OUR (kg/m^3.d) equation follows:

$$OUR = \frac{(DO_{in} - DO_{out})Q_r}{V_b} \qquad \text{Equation 4-1}$$

Where DO_{in} and DO_{out} are the concentrations of DO entering and exiting the filter, respectively, Q_r is the flow rate through the filter, and V_b is the volume of beads (Malone and Beecher, 2000). The recirculation flow rate is critical in maintaining acceptable levels of OUR. Figure 4-7 demonstrates the negative correlation between OUR and effluent $CBOD_5$ concentration exiting the filter bed (r=-.793, significant at the 0.01 level). Shaded data markers denote oxygen limiting conditions, defined by effluent concentrations from the media bed containing less than 1.0 mg/L of dissolved oxygen. Oxygen limiting conditions transitionally occurred at high organic levels in the media bed after a heavy loading event, causing complete impairment of the filter. Oxygen limitations are not limited, however, to high organic concentrations inside the media bed and low OUR values. Regardless of the organic conditions, insufficient oxygen concentrations cause at the least a partial impairment of biological activity in SLDM filters. While 16.7% of the data points collected for the BF4 system were found to be oxygen limited, 48% of the BF6 data experienced oxygen limitations.

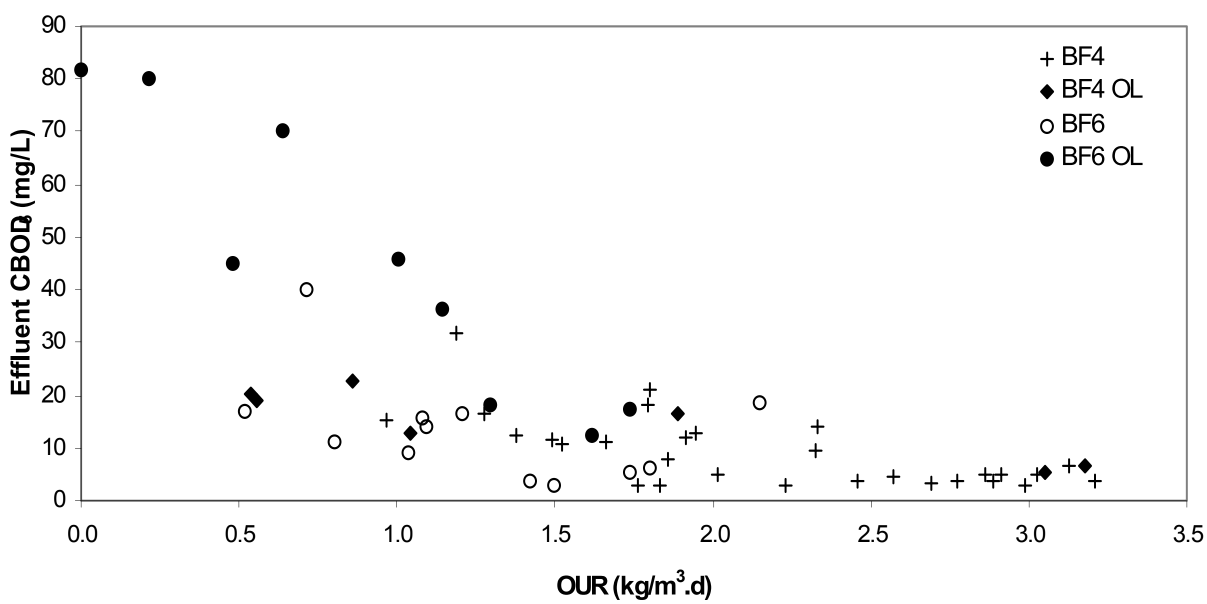


Figure 4-7. Higher oxygen uptake rates (OUR) are associated with decreased organic levels in SLDM filter effluents.

The relationship between dissolved oxygen concentrations in the filter and $CBOD_5$ reduction was used to evaluate filter performance. Although a complex function of microbial population, loading to the filter, solids removal efficiency, and operational parameters, a generic, simplified relationship was used to evaluate overall performance at different bed concentrations. The strategy relates the dissolved oxygen uptake rate of the biofilm to the removal rate of $CBOD_5$ (BOD_r, in $kg/m^3.d$) applied to the filter by a function labeled the MX Factor. The removal rate of BOD (BOD_r) is expressed in Equation 4.2.

$$BOD_r = \frac{(CBOD_{in} - CBOD_{out})Q_r}{V_b} \quad \text{Equation 4-2}$$

Where $CBOD_{in}$ and $CBOD_{out}$ are the concentrations of $CBOD_5$ entering and exiting the filter, respectively, and Q_r and V_b are as previously defined. Recirculation flow rate is also critical in maintaining high levels of BODr. Dissolved organic removal rates and oxygen kinetics are known to be stochiometrically related under steady state conditions using a respiratory quotient. The relationship used in this study between these factors is expressed as the dimensionless MX factor.

$$MX = \frac{BOD_r}{OUR} \quad \text{Equation 4-3}$$

During typical operation, in which the filter bed is not substrate limited, MX values may range from 2 to 3, and may be higher if high levels of reaeration are achieved (Wagener *et al.*, 2002). Hu *et al*. (1994) investigated the oxygen uptake characteristics of a submerged biofilter employing different sized media sunken media. That study found when BOD removal was low, endogenous respiration accounted for a large part of the OUR. During periods of very low BOD concentration in SLDM filters, MX values were less than one, which was attributed to endogenous respiration of the bacteria, although ratios below one may have also indicated that some nitrification was occurring.

HYDRAULIC LOADING

The hydraulic flux , or filtration rate, is directly proportional to the water flowrate through the filter, which is a parameter that can be adjusted to manage solids removal. According to Ahmed (1996), the principal factor influencing solids removal in SLDM filters is the hydraulic flux. Filtration rate is inversely related to single pass efficiency, the percent reduction in TSS. It is also directly correlated to the overall capture rate (mass of TSS retained), as increased solids transport more than compensates for the reduction in per pass efficiency. Single pass removal efficiencies vary with particle size; however, in a recirculating format even the fine solids are quickly removed since the water typically passes through the filter multiple times per hour. Thus for single pass applications, lowering the flowrate enhances removal efficiency (Liao and Ødegaard, 2002; Ahmed, 1996); however, in recirculating systems the lowest TSS levels are obtained by maximizing the flowrate (Ahmed, 1996).

Table 4-3. Solids Accumulation for Associated Hydraulic Application Rates, from Stensel *et al*., 1988.

Filtration Rate (m/h)	Solids Accumulation (kg TSS/m^3 media)
3.7	2.4 (2.1-2.6)
2.9	2.6 (2.1-3.3)
2.4	2.5 (2.2-2.7)
1.6	3.0 (2.6-3.2)
1.3	3.3 (3.2-3.6)

In a full scale BAF plant evaluated by Stensel *et al*. (1988), the filtration rate was found to be inversely proportionate to the solids accumulation by the media, as shown in Table 4-3.

The quantity of media was the same for each filtration rate, thereby providing an equal basis for comparison of flux rates.

The removal of suspended solids was an important consideration in SLDM filters when aiming to reduce total $CBOD_5$ concentrations for single and multi-pass regimes. The fraction of particulate bound $CBOD_5$ in the wastewater was intermittently tested during filter evaluation, and was found to increase from 0.37 to 0.73 as the wastewater traveled from the influent point until it exited from the filter, indicating that increased solids capture could help to reduce the levels of $CBOD_5$ in SLDM filters. Filtration rates in the SLDM filter prototypes tested ranged between 5 and 26 m/h, and increased filtration rates were associated with lower TSS and $CBOD_5$ effluent concentrations in both BF4 and BF6 systems. Table 4-4 compares the filtration rates used in the BF4 and BF6 experimental units with other upflow granular medium filters employing low density media.

Table 4-4. TSS removal based on media type and filtration rate in upflow floating granular medium filters.

Floating Media	Apparent Filtration Rate (m/h)	% Removal TSS	Coagulant	Study
EN*	10-15	80-95	No	BF4
	10-15	50-90	No	BF6
Ring Shaped Media	42	80	Yes	(Tanaka et al., 1995)
Kaldness Biofilm Carriers	2.5 - 10	80 - 90	No	(Ødegaard et al., 1998)
	5.0 - 10	78 - 87	Yes	(Ødegaard et al., 1998)
	10-15	69 - 85	No	(Ødegaard et al., 1999)

* The EN media was tested in two different prototypes.

The filtration rates in this study were similar to rates found in other floating media filters (Ødegaard and Helness, 1999; Sampa and Tanaka, 1995). In a study by Ødegaard and Helness, the highest removal rates were achieved at the lowest filtration rate tested (10 m/h) (1999).

Tanaka *et al*. found that suspended solid removal rates dropped markedly when filtration rate exceeded 42 m/h (1995). Removal rate continued to improve with decreasing flow velocity to 33 m/h, the lowest filtration rate tested in that study (Tanaka *et al*., 1995).

Tanaka *et al.* (1995) used a polyelectrolyte coagulant in conjunction with a ring shaped floating media 10-15.2 cm in diameter by 2.5 cm in length. TSS removal rates for the ring shaped media averaged at 78%, corresponding to an effluent concentration of 34 mg/L, prior to filter breakthrough in a pilot scale filter dosed with at 2.5 mg/L polyelectrolyte concentration.

TSS removal rates of 84.5 to 69.1%, corresponding to an effluent concentration of 30-35 mg/L of TSS, were achieved for various media sizes and filtration rates in a study by Ødegaard and Helness (1999). The two biofilm carriers tested were KMT media; the smaller carrier was 7 mm in length and had a diameter of 10 mm, while the larger media was 15 mm in length and diameter. The experimental plant evaluated by Ødegaard and Helness consisted of a MBBR followed by a floating media filter. The MBBR was assumed to have little impact on the solids removal capacity, other than the fraction of particulate organics that were hydrolyzed (Ødegaard and Helness, 1999). The study by Ødegaard investigating the impacts of coagulant use in floating medium filters used KMT media with a diameter of 7 mm and a length of 8 mm.

Flow Variation

Most wastewater treatment facilities receive variable flow. Although the pattern tends to repeat itself on a daily basis, a constantly changing pattern can cause failure in some wastewater treatment systems. Recirculating biofilters have been found to be resistant to flow variations, yielding reliable effluent in spite of wide flow variations (USEPA, 2002). The flow received by the bioclarifiers used in this study varied naturally according to uses at the study site.

Media Bed Properties

The selection of a suitable medium is critical in the design and operation of SLDM filters. A number of physical factors that influence biofilm growth are dictated by media properties. The shape, size, and density along with surface roughness affect the flow and resulting shear stress on the biofilm, as does the available surface area for biofilm attachment (Mann and Stephenson, 1997).

MEDIA SIZE

Different sized media may be preferable for different applications in SLDM filters. Guidelines for media sizes in BAFs, in which largest medium sizes were recommended for roughing filter applications, intermediate sizes for secondary treatment, and smaller sizes for tertiary treatment (Moore *et al.*, 2001), may be applicable to SLDM filters. Furthermore, additional applications of SLDM filters, such as pure solid-liquid separation functions may require multiple sizes of media. To date, sizes of media tested in SLDM filters have ranged from 10 cm in diameter to 50 microns in diameter.

Stensel *et al*. (1988) found that when smaller media (2.8 mm diameter) was used, lower average hydraulic application rates and loadings could be used to achieve similar effluent quality with larger media (4.4 mm diameter) in a high organic loading BAF cell. That is, retention time had to be increased (from 26 to 44 minutes) and volumetric organic loading decreased (from 5.6 to 3.3 kg $BOD_5/m^3.d$) when using larger media. Smaller media have been found to require more frequent backwashing than biofilters employing larger media (Moore *et al.,* 2001). In biofilters with sunken media, more pronounced SS overshoot effects were seen following backwashing of larger media as compared to the smaller media (Moore *et al.*, 2001). If too small of a media is used, flow will not distribute uniformly in the filter bed and short circuiting of water may result in incomplete treatment (Mouri and Niwa, 1993). Previous studies in SLDM filters have indicated that suspended solids removal increases with a decrease in size of individual media (Ahmed, 1996).

MEDIA SPECIFIC SURFACE AREA

Specific surface area (SSA) of a biofilm carrier is the ratio of the surface area to the volume that a media occupies. Comparisons between biofilters employing different sizes and shapes of biofilm carriers have shown similar organic removal results when compared on an areal basis (Ødegaard *et al*., 2000). Biofilters, regardless of the media characteristics, will achieve similar biological conversion results if the surface area available for biofilm growth is the same on the media. By consequence, biofilters may become smaller as the SSA of the filter media increases. When the SSA of the media changes, however, volumetric organic removal rates and OURs change as well. Biofilters employing a fixed volume of media may reduce the required retention time by choosing a media with an increased SSA (Meunier and Williamson, 1981).

The effective surface area available for aerobic biofilm growth is an additional parameter that should be considered when selecting a media for use in biofilters. Optimally the biofilm should be protected from excessive biofilm detachment, however surface area provided by intricately shaped beads that offer several niches for growth and high clean bed SSAs may ultimately become useless after acclimation. Previous studies comparing the nitrification performance of spherical and tubular buoyant media in SLDM filters found that increased organic loadings resulted in clogging of the tube shaped medium, reducing the SSA to less than 50% of its original clean bed value (Sastry, 1996).

MEDIA DENSITY

Along with a static bed of media, another defining characteristic of SLDM filters is the use of floating, or low density, media as biofilm carriers. As such, the impact of density differences in SLDM filters has not been tested, and the use of varying density media is limited to dual medium filters in which each media having specific gravities of less than one and two layers of differently sized or shaped media could be created. Biofilm carriers of various densities have been tested in BAFs and higher levels of soluble COD were found to be removed in filters containing floating media rather than sunken media, while the filters maintained similar conditions, including media size and shape (Mann *et al.,* 1999; Mann and Stephenson, 1997). Solids removal has also been shown to be more positively effectuated by lower density media in BAFs, 80-90% removal for floating media as compared to 50-60% with sinking media (Mann *et al*., 1999).

FLOW DIRECTION

When the filtration mode is based upon downflow operation, pressure on the bed can not increase significantly, as localized fluidization will induce bypassing in sections of the bead bed. During normal operation, the buoyant forces maintain the media in a packed configuration, overcoming the sum of the drag force exerted by the flowing water plus the weight of the beads and retained solids. Fluidization occurs when the fluid drag increases to the point at which this occurs, which has been observed to occur at flux rates of less than 12 m/hr (5 g/min.ft^2) in downflow SLDM fitlers using spherical low density polyethylene media of 3 to 5 millimeters in diameter (Ahmed, 1996). Solids capture in a downflow bead filter is deleteriously impacted by fluidization of the filter bed (Ahmed, 1996).

SLDM filters can be configured for operation in either an upflow or downflow mode. The filters used in this study were operated under upflow conditions. Previous studies have shown that during upflow operation, SLDM filters containing polyethylene beads with a specific gravity of 0.89-0.91 could handle high fluxes (greater than 73 m/hr (30 g/min.ft^2)) with substantial solids capture (Ahmed, 1996). The high filtration rates offered by an upflow filter are beneficial when attempting to maintain aerobic conditions in the filter bed in a recirculating SLDM Filter. In either operating mode, the cleaning frequency required to restore sufficient hydraulic conductivity is directly determined by the organic and solids loading rates applied to the filter.

Substrate Loadings

Both organic and solids loadings express a positive linear relationship with their corresponding removal rates. The organic loadings applied to the BF4 and BF6 systems were in the range of ranged from 0.3 - 3.8 kg/m^3.d in the BF4 system and from 0.6 – 4.6 kg/m^3.d in system BF6. The total volumetric solids loadings applied to the systems also varied, ranging from 0.5 to 2.4 and 0.3 to 3.0 for systems BF4 and BF6 respectively. Figure 4-8 illustrates the organic removal results, in terms of applied and removed loading of $CBOD_5$ to the SLDM filter system for experimental units BF4 and BF6, similarly the graphs in Figure 4-9 are for the TSS loads on the experimental units. Oxygen limiting conditions were defined to have occurred if the effluent DO from the filter bed was less than 1.0 mg/L. As Figure 4-8 reveals, during periods of oxygen limitation, the percentage removal of organic matter typically decreased. Solids loadings were also found to elicit a linear removal in SLDM filters when properly operated. A breakdown in the linear relationship was seen beyond loadings of 1 kg/m^3.d in BF4, and is attributed to improper design and maintenance of the filter.

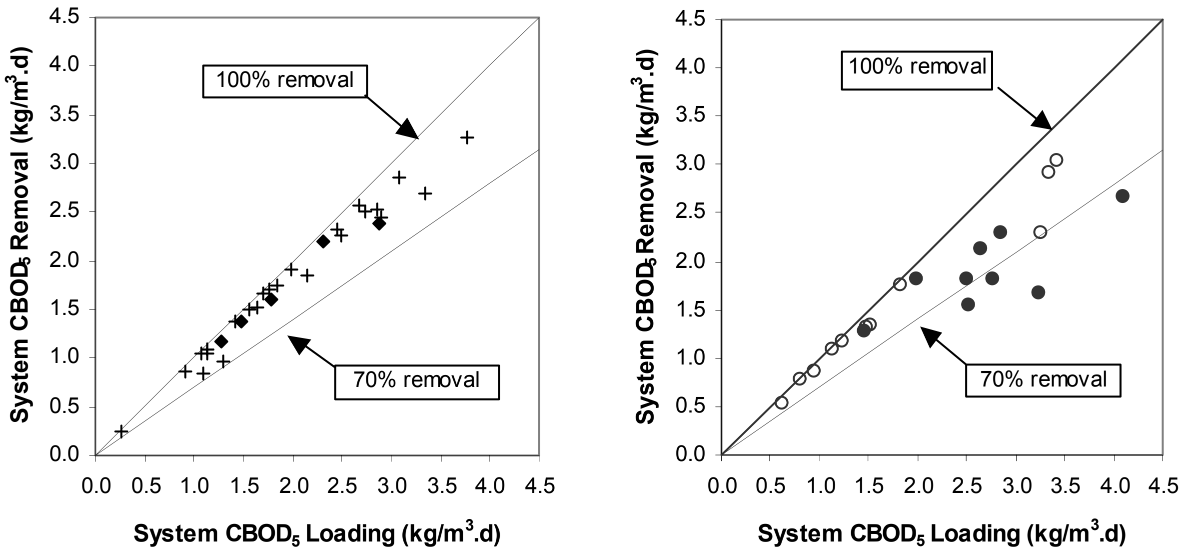


Figure 4-8. When properly operated, SLDM filters may obtain organic removals of greater than 70%, as seen in filter BF4. Oxygen limitations (indicated by filled in data markers) as well as impaired backwashing ability affected organic removal at higher loadings in the BF6 system.

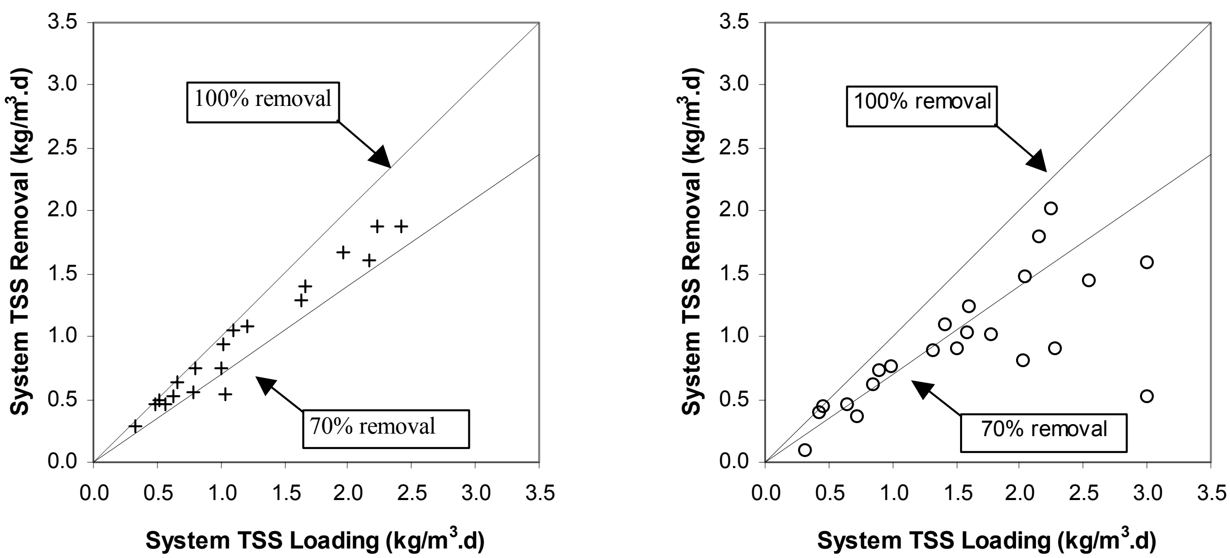


Figure 4-9. Solids removal and loadings were found to be linearly related as shown on the left for the BF4 system. In BF6, on the right, a breakdown in the linear relationship was seen beyond loadings of 1 kg/m3.d is attributed to improper design and maintenance of the filter.

Conclusions

Several parameters have been identified as manageable influences on the performance of SLDM filters, including: backwashing parameters, recirculation flow rate, properties of the media bed, and loading rates. An effective backwash strategy in terms of backwash intensity and backwash frequency should be incorporated into the design and operation of SLDM filters. Proper design of filters may limit the dirty washing effect, improving TSS effluent quality. Frequent washing is necessary to maintain appropriate level of aerobic conditions, organic removal, and solids capture in SLDM filters. As SLDM filters minimize water loss, backwash frequency may be utilized as a means of controlling these parameters as well as biofilm qualities.

Recirculation flow is a critical parameter for the maintenance of appropriate residence times within SLDM filters, as well as oxygen levels and filtration rate. This both directly and indirectly impacts biological conversion and solids capture. Short residence times of 40 minutes, similar to required residence times in other biofilters, was found to be sufficient for an 85% removal of $CBOD_5$. Batch studies revealed the majority of solids capture and organic removal occurred within the first hour in SLDM filters. Oxygen uptake rates (OUR) of 3 $kg/m^3.d$ were found to be correlated with low $CBOD_5$ effluent concentrations. Filtration rates of 10 to 15 $m^3/m^2.h$ were found to provide acceptable solids removal.

SLDM filter operation is affected by additional properties related to the operation of media beds. Upflow direction was found to be preferable in SLDM filters, as well as floating biofilm carriers that provide high specific surface areas. Size of the media used in SLDM filters was also identified as an important parameter that may be adjusted according to application. System loadings and removal rates of organic material and suspended solids were found to have a linear relationship when properly designed and operated. When operated with insufficient dissolved oxygen and impaired backwashing, organic and solids removal deviated from a linear relationship when more highly loaded.

CHAPTER 5 : GLOBAL DISCUSSION AND CONCLUSIONS

The Static Low Density Media (SLDM) filter may be effectively used as a bioclarifier to treat domestic wastewater. Combining a SLDM filter with an airlift pump and a recirculation basin, when operated the packed bioclarification mode with a low headloss, results in an effective treatment strategy with the functions of concurrent aeration, biofiltration, and clarification all being effectuated by an air blower, recirculating tank, and a floating media filter. Results from this study indicate that such a system could adequately provide secondary treatment for domestic wastewater, and could replace more traditional treatment operations and processes, such as an activated sludge basin followed by a secondary clarifier, or a trickling filter followed by its associated clarifier. Furthermore, use of SLDM filters follows the consolidation strategy of combining multiple functionalities in a single unit, and may result in savings of capital and operational costs.

The SLDM filters were found to require little maintenance, provided the units were properly designed and operation was kept within general guidelines for backwash frequency, recirculation flow rate, and substrate loadings. This advantage of the SLDM filter broadens its potential application to include both large and small scale installations. In locations with high water tables, such as state of Louisiana where an estimated fifty percent of the over four hundred and forty thousand onsite treatment systems have been reported to fail (USEPA, 2002), use of SLDM filters could positively contribute to the improvement of local water quality. Other applications of SLDM filters include remote facilities and locations without highly trained personnel.

This study aimed to develop design and operational values for SLDM filters used to both physically and biochemically treat secondary domestic wastewater. These preliminary values are listed in Table 5-1 for systems with a BF4-type configuration and for the effluent criteria of both 10 and 20 mg/L of $CBOD_5$.

Table 5-1. Proposed design values for SLDM filters.

Parameter	10 mg/L $CBOD_5$ Effluent	20 mg/L $CBOD_5$ Effluent
Organic loading (kg $CBOD_5/m^3$.d)		
Bed Loading	15	25
System Loading	2.7	4.0
Solids loading (kg TSS/m^3.d)		
Bed Loading	12	28
System Loading	1.3	2.4
Backwash Interval (hours)	2.5 to 6	
Flow Ratio (Q_r/Q)	Greater than 60	
Total Retention Time (minutes)	30 to 60	
Filtration Rate (m/h)	10 to 15	

* Bed Loadings indicate single pass loadings while system loadings indicate multiple passes.

Organic Loading

Operation of a biological system can be evaluated by determining the upper limits of organic loading that the system can handle. Typically, a more heavily loaded unit will result in higher effluent concentrations. Organic loading to the entire SLDM filter system, as previously described, was calculated via Equation 5.1:

$$Loading_l = \frac{S_{si} * Q}{V_b} \quad \text{Equation 5.1}$$

Where S_{si} is the substrate, namely $CBOD_5$, applied to the system, Q is the flow rate through the entire system, and V_b is the volume of the filter media. The bed loading rates, as calculated from Equation 5.2, were determined through measurement of S_{ri}, the substrate, $CBOD_5$ or TSS, entering the filter bed, and Q_r is the recirculation flow rate through the filter

$$Loading = \frac{S_{ri} * Q_R}{V_b} \quad \text{Equation 5.2}$$

Organic loadings and their corresponding effluent concentrations were determined and compared to alternate biological systems. The location of SLDM filters within the spectra of domestic wastewater treatment systems in regards to its capacity for biological treatment can be summarized by a comparison of organic loading capacities. When compared to the organic loadings used to achieve a 90% reduction in BOD for other biological treatment processes, SLDM filters were found to be highly effective. Table 5-2 summarizes these results.

Table 5-2. Organic loading capacity of SLDM filters indicate highly effective organic conversion compared to other bioreactors.

Reactor Type	Organic Loading $kg/m^3.d$		Source
Trickling Filter	0.1 – 0.4	BOD	(Mann *et al*., 1995)
Activated Sludge	0.3 – 0.6	BOD	(Mann *et al*., 1995)
BAF	0.7 – 2.8	BOD	(Mann *et al*., 1995)
SLDM Filter	2.0 – 2.8	$CBOD_5$	

To achieve a 90 percent reduction of $CBOD_5$, the maximum organic loading applied the BF4 and BF6 systems was found to be 2.8 and 2.0 $kg/m^3.d$, respectively. Bed loadings, not recorded in the above table, are higher. Additional studies on BAFs have generated data pertaining to achieving specific effluent concentrations based on organic loading. In a two year full scale BAF study by Stensel *et al*. (1988), the highest organic loading used was 5.0 kg $BOD_5/m^3.d$ to achieve effluents of less than 30/30 mg/L of BOD_5 and TSS. Typical organic loading levels for BAFs have been reviewed and recorded by Mann *et al.* (1999), to be 2.5 kg $BOD_5/m^3.d$ and up to 18 kg $COD/m^3.d$. In the BF4 unit, applied loads of 2.7 and 4 $kg/m^3.d$ of $CBOD_5$ should result in effluent concentrations of 10 and 20 mg/L, respectively.

Solids Loading

Treatment in SLDM filters was found to be largely dependant on solids removal. The fraction of particulate bound $CBOD_5$ in the wastewater was intermittently tested during filter evaluation and found to increase from 0.37 to 0.73 as the wastewater traveled from the influent point until it exited from the filter. The relationship between TSS and $CBOD_5$ in the effluent of SLDM filters reveals that a higher TSS removal would lower $CBOD_5$ concentrations.

Loadings applied to SLDM filters were calculated via Equations 5.1 and 5.2, replacing TSS as the substrate concentration. Effluent TSS concentrations of 10 mg/L were achievable at solids loadings of 1.3 and 0.4 for the BF4 and BF6 systems. Corresponding one pass bed loadings were found to be 12 and 9 $kg/m^3.d$

Operational Parameters

Batch studies revealed the majority of solids capture and organic removal occurred within the first hour in SLDM filters. Oxygen uptake rates (OUR) of 3 $kg/m^3.d$ were found to be correlated with low $CBOD_5$ effluent concentrations. Filtration rates of 10 to 15 $m^3/m^2.h$ were found to provide acceptable solids removal. Poor organic and solids capture effectiveness in SLDM filters may be due to design, management, or intrinsic filter inadequacies. Design deficiencies included the arrangement of the sludge storage area. Management problems include infrequent or incomplete filter backwashing, and improper recirculation flow rates. Intrinsic filter inadequacies include undersized filter beds. With the exception of under sizing the filter, most problems leading to high effluent concentrations can be easily corrected. Furthermore, the particular filters tested in this research effort were subjected to widely varying hydraulic conditions that required larger volumes for recirculation basins. Developing upstream hydraulic dampening would reduce the size requirement of the recirculation basin component as part of the overall system design.

Recommendations

Additional testing on SLDM filters could better demonstrate their capacity to perform as a secondary treatment unit. Media selection, backwash frequency, and stabilization of influent flow are all parameters that require additional information before this technology could be widely applied. If more constant wastewater was applied, the recirculation tank size could be minimized, thereby reducing the footprint and cost of the system. In this study, the characteristics of the site required a large recirculation tank, as no wastewater flow was generated over the weekends, nights, and holidays. When the system did receive influent it was typically in large slugs. Further evaluation of these filters may reveal the impact of this configuration, specifically, if larger organic loadings would be achievable under a more traditional flow regime.

Higher organic loadings may be obtainable with optimized media usage in SLDM filters. In addition, operational problems such as clogging when SLDM filters are not protected by primary sedimentation may be solved through media selection. A courser and slightly more dense (although still floating) media could be used as a bottom layer in an upflow dual media SLDM filter to remove larger particulates The combination of two media into a dual media filter has previously been recognized as having a great potential for optimizing bed configuration in order to decrease head loss considerably with acceptable suspended solids removal (Liao and Ødegaard, 2002).

Layout of the filters in a series or array could also enhance the performance of multiple units, and would also reduce the impact of the SS Overshoot following backwashing events. SS Overshoot control should also be investigated through testing of different backwashing designs and through management techniques. Reducing the filtration rate following a backwash may be one means of controlling TSS concentrations during the recovery period for SLDM filters.

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APPENDIX A: EXPERIMENTAL REPORTS

Final Report: BF4 Prototype

Experimental Report Number 2002.01

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November 21, 2002

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Institute for Ecological Infrastructure Engineering

Prototype 4 Experimental Report

Abstract

From June 2001 to January 2002, the Delta 4 Experimental Unit was in operation at the Delta Environmental Test Site located in Denham Springs, Louisiana. During this period, the system was operated and evaluated at various hydraulic and organic loading rates, recirculation rates, backwash intervals, and ambient temperatures. Analytical work was conducted in triplicate according to Standard Methods for the Examination of Water and Wastewater and the averaged results are presented (APHA, 1995).

Introduction

The fundamental design of most large scale domestic wastewater treatment plants is a classical unit operation configuration, consisting of primary clarification, biological treatment, and secondary clarification. This approach is effective, but results in treatment plants requiring a large footprint, high capital resources, and sophisticated staffs. Smaller communities may not have the resources for a large centralized plant and may instead opt for individual on-site treatment systems. Regardless of the means, these communities demand technologies that are robust and relatively simple to operate without any compromise of effluent quality. Such approaches can be developed systematically by consolidating the treatment train through utilization of technologies that can perform more than one process. A treatment system designed within these parameters could provide both large and small communities an attractive alternative to current systems.

Background

Static low-density media (SLDM) filters are known in the aquaculture community as Floating Bead Filters (FBFs). The units are currently widely employed as clarifiers or bioclarifiers in support of high-density recirculating production and holding systems for fish, reptiles, and crustaceans (Malone and Beecher, 2000; DeLosReyes and Lawson, 1996). Historically, SLDM filters have been used exclusively in aquaculture applications, therefore; most research has been in improving nitrification capacity, as that has been identified as the limiting factor in bioclarifier performance for aquaculture applications (Sastry, 1999; Malone *et al*., 1993). SLDM Filter technology has not been applied to domestic wastewater treatment for concurrent biological and physical treatment. Technologies that have been employed in domestic wastewater treatment capacities similar to SLDM filters include: biological aerated filters (BAFs), trickling filters, and sand filters.

SLDM Filters are fixed-film filters, typically operated in an upflow configuration, in which a biofilm support medium is submerged in wastewater to create a large contact area for aerobic biological treatment. Air or oxygen is supplied to the filter without disturbing the media, resulting in a static bed and necessitating recirculation of the wastewater in order to maintain aerobic conditions. Airlift pumps can readily provide submerged packed beds challenged with elevated BOD levels the necessary high rate of recirculation and aeration that they demand (Loyless and Malone 1998; Reinemann and Timmons, 1989). The combined floating media filter and airlift unit, when operated the packed bioclarification mode with a low headloss, results in an effective treatment strategy with the functions of concurrent recirculation, aeration, biofiltration, and secondary clarification all being effectuated by an air blower, recirculating tank, and a floating media filter.

The low-density plastic media acts as a carrier for biofilm and as a physical separation mechanism for solids. Heterotrophic bacteria attach themselves to the beads and utilize the organic matter in the waste stream as a carbon source for growth, while autotrophic, nitrifying bacteria convert ammonia to nitrate under conditions of low organic loading (Zhang et al, 1995). Concurrently, suspended solids in the waste stream are captured in the bed via surficial straining, deep bed filtration, and adsorption as the waste stream travels upward though the bead bed. The capture of solids in a SLDM Filter is known to be influenced by particle size, filter media size, flowrate, and solids accumulation (Ahmed, 1996).

The SLDM filter used in this study supplied oxygen through recirculation of the wastewater through the filter bed by means of airlift pumps. The wastewater made multiple passes through the filter bed with retention times of 30 seconds to one and a half minutes per pass. At the end of each pass, an airlift pump would return the wastewater to the equalization basin chamber. The airlift pump served the dual functions of water delivery and aeration.

The beds of SLDM filters are periodically expanded for removal of accumulated solids and excess biofilm (Malone and Beecher, 2000; Cooley, 1979). Backwashing or expansion of a bead bed can be accomplished by hydraulic, pneumatic, or mechanical means. SLDM Filters take advantage of the buoyancy of the media to minimize the water loss that would otherwise be associated with the high frequency washing needed to manage the biofilm. These units are capable of restricting water losses to periods of sludge removal, as opposed to other filters which can use ten percent of the product water (Stensel *et al.,* 1988) to twenty percent of the influent wastewater (M'Coy, 1997) to hydraulically wash the media. SLDM Filter units are virtually impervious to caking problems that can plague granular filters that are subject to high organic loads. Since backwash water loss is minimal in filters with internal propellers and in the drop filter units, the backwash frequency can be employed as a biofilm management tool (Malone *et al*. 1993). Additional biofilm management flexibility is obtained by altering the bead shape or by moving to a less aggressive washing format (Golz *et al*. 1999). However once the unit is selected, backwash frequency is the principle operation parameter used to enhance biofiltration performance.

Methods and Materials

The Delta 4 prototype filter was configured in the manner shown in Figure 1. The prototype received flow from a large, 1000 gallon tank that acted as both storage and as a primary clarifier. This was followed by the Delta 4 Experimental Unit, which consisted of a bead filter and a recirculation tank. The filter was two feet in diameter and four feet tall, and contained 42.5L (1.5 ft^3) of biofilm carrier. The carrier was a modified media 3 to 5 mm in diameter, with a density of 0.90 kg/L, a porosity of 0.55, and with a total specific surface area of 1100 to 1250 m^2/m^3 (Malone *et al*., 1993). The recirculation tank had a 1.42 m^3 capacity (four feet in diameter and four feet tall). Following the recirculation tank was an effluent holding tank. This effluent sump contained a trash pump and was followed by a meter so that the total volume exiting through the system could be determined.

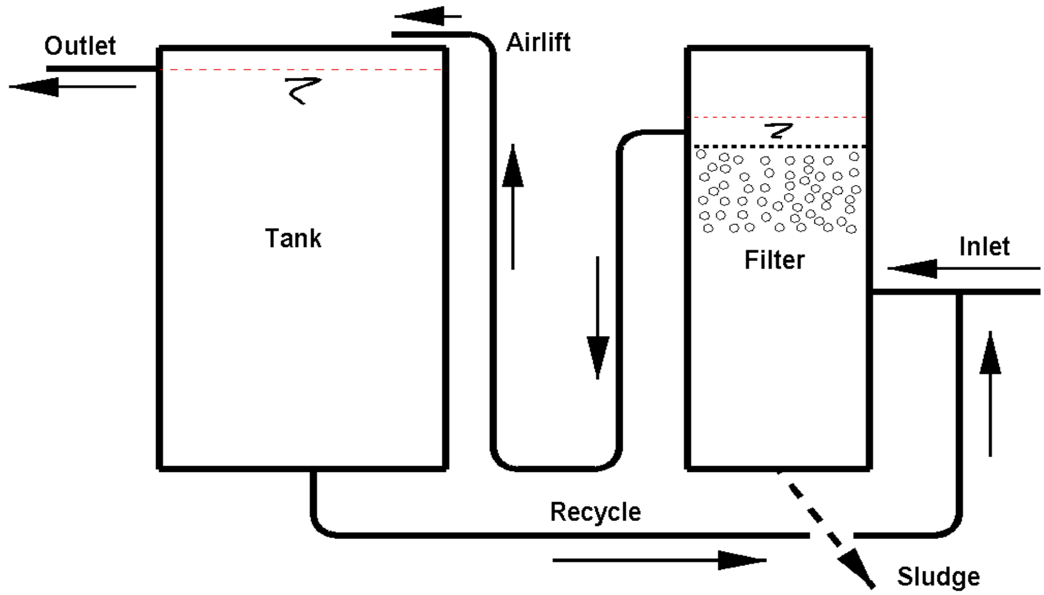


Figure 1. General Configuration of Delta Prototype 4.

The site was subject to highly variable flow characterized by morning and afternoon peaks. Temperature, pH, and flow measurements were recorded along with other operational parameters, such as backwash frequency, during each sampling event. The sampling procedures are outlined in the section below. Water quality parameters were tested in triplicate in the Water Quality Laboratory located at Louisiana State University, according to Standard Methods for the Examination of Water and Wastewater and include the following: $CBOD_5$ (5210B), DO (4500-O C), TSS (2540 D), and VSS (2540 E) (APHA, 1995).

Table 1. Influent Wastewater Characteristics

Parameter	Mean Value
$CBOD_5$, mg/L	104
(n)	(33)
TSS, mg/L	61
(n)	(20)
Temperature C	23.3
(n)	(37)

The entire system was operated for more than one month in an acclimation mode, prior to testing. During this period wastewater was circulated through the filter, but the backwashing frequency was lowered so that bacteria could populate the biofilm carrier.

Sampling Procedures

Upon arrival at the Delta Environmental Test site, the date and time was recorded and then a quick scan of the site commenced. The *Backwash Indicator Line* was checked to determine when the filter would next backwash, and the sampling time was determined accordingly. Samples interrupted by a backwashing event were considered not usable. If a Backwash occurred while at the site, the time was recorded.

In situ measurements were then recorded. The number from the System Effluent Meter was recorded. This meter displayed the volume exiting from the filter and recirculating tank. It should be noted that one revolution on this meter, was ten (10) gallons and that this point may not have been clear in previous Delta Experimental Units. The temperature in the recirculating tank was also recorded with a glass mercury thermometer. Other observations concerning the site and the filter appearance were also recorded.

The System Out sample (i.e. effluent from the entire system) was taken first. Typically two liters were taken for BOD and TSS analysis. For the majority of the experiment, this sample was taken in two 1 liter plastic bottles, and every effort was taken to assure that the sample was well blended. From December onward, this sample was taken with a single 2 liter plastic bottle. If the filter had recently backwashed, this forced the sampler to wait until the filter produced overflow, the effluent.

The Recycle Out and Recycle In samples were taken next (flow out of and into the bead bed, respectively). Two liters of sample were taken in 2 L plastic bottles for BOD and TSS analysis, and three BOD bottles were filled with each sample for DO analysis by Winkler Titration. An additional sample volume was taken (1 L) if a DO meter was used. The sample bottles were first partially filled, then the BOD bottles were filled. During this process, no air bubbles were allowed to travel through the sample lines, otherwise the sample was rejected and retaken. The BOD bottles were filled by inserting the tube into the bottle so that it touched the bottom of the bottle, and the bottles were allowed to overflow before the tubing was removed. After all the BOD bottles were filled, the sample bottles were filled to capacity and allowed to overflow. After the samples were taken, the reagents used in the Azide modified Winkler DO procedure were added, in preparation for titration at the Water Quality Laboratory.

The recirculation flowrate was then taken via a bucket with an eight liter mark and a stopwatch. The bucket was allowed to sink as it filled to maintain a constant level of water in the tank. This measurement was taken three times. This sample was not taken before any of the samples were taken, as it was observed that this measurement procedure disturbed conditions inside the tank and possibly caused mixing, which could have given a sample with falsely high contaminant concentrations. If a backwash occurred after this point, the samples were considered still useable, however, if a backwash occurred anytime between collection of the System Out sample and this point, the samples were be discarded and these steps must be repeated once there is overflow into the final sump.

The headloss through the filter was then measured with a ruler in units of centimeters. Four different headloss measurements were recorded: Top Screen, Bed, Bottom Screen, and Total. These are measured by the following tube combinations: top two, middle two, bottom two, and the top and bottom tubes, respectively. The bottom screen was typically less than 1 cm, which was recorded as < 1 cm. If a backwash occurred after the recirculation flowrate was measured, the head loss could not be recorded.

The Observed Backwash Interval was then measured. A stopwatch and ruler were used to determine the time it took for at least 1 cm of water to be displaced in the Backwash Indicator

Line (BIL). The BIL was simply a piece of tubing connecting two holes, drilled at the top and bottom of the backwash air chamber. It was assured that the BIL was not clogged with beads or other material, which could give false readings.

The System In sample was then taken in a 1 L plastic bottle. The first 100 to 400 milliliters were discarded, as it contained large solids that settled into the fitting. Then the bottle was filled to near capacity. The samples were placed into an ice chest and put on ice for transport back to the Louisiana State University Water Quality Laboratory.

The pH of the samples was then checked with a portable pH meter, if it was brought to the site. The pH was checked in the sample bottles moving from least concentration to most and the probe was washed between bottles with DI water.

Sludge was discarded at least once a week until the effluent was no longer dark (about 1 to 2 gallons).

After sampling ended, the sampler then made additional observations around the site where the Delta 4 prototype equipment was located and checked to verify that everything looked normal and was operating functionally. If the BIL had beads or any obstructions in it, the line was opened to flush them out. If the Backwash Frequency needed alteration, it was done at this point, and the new interval was recorded. Also, if the pressure gauge on the backwash air line was reading a value above 80, the pressure in the line was reduced. The new backwash frequency was then checked and any changes and the new interval was then recorded. The sampler also was to look in the bed window and make observations about the beads and biofilm, noting any unusual color or macroorganisms. Finally, the sampler was to make sure that the cover for the recirculating tank was on so that rain and sunlight was blocked. This was part of the continuous effort to prevent algae from growing in the recirculating tank

Results

Experimental results for the BF4 evaluation period from July to January 2002 have shown carbonaceous biochemical oxygen demand ($CBOD_5$) concentrations to decrease from 104 mg/L to 9 mg/L on average for this period and through multiple passes of the filter. Total suspended solids (TSS) concentrations have been shown to decrease from 61 mg/L to 9 mg/L on the average. During the sampling period, field visits were made on 63 days, of which field sampling for at least one analytical parameter occurred 60 times. Some sampling events resulted in multiple samples for variable times. The entire collected data set can be found in Appendix B. It can be seen that not every data set in Appendix B is complete or useable; this is attributed to either issues in the field or a failure to meet criteria in laboratory analysis.

Two notable field issues were a chemical interference in the system and an air intrusion in the filter. The chemical interference occurred in July of 2001; it was discovered during analysis of dissolved oxygen that a pinkish-brown gas, suspected of being iodine, was generated in the head space of the bottles containing sample and the Winkler reagents. This chemical was found to interfere with the titration involving sodium thiosulfate in the Winkler procedure, resulting in inflated dissolved oxygen concentrations. The problem was remedied on the short turn by measuring dissolved oxygen both with the Winker titration method and with a dissolved oxygen probe. Eventually, the chemical interference ceased to appear in the samples and the two methods of analysis reached agreement. It was assumed that the chemical causing the interference was no longer present after this point. As the preferred method of dissolved oxygen analysis was the Winkler Method, those results were used whenever the probe results were in agreement.

The second notable field issue was air intrusion into the filter. Air was collecting above the top plate of the filter, which resulted in air bubbles in the "Recycle Out" sample line. The problem was first recorded in August of 2001, and it was determined that there were two contributing sources. Air from filter backwashing was collecting above the top plate and was not completely forced out, as well as air from outside of the filter, which was entering from several pin hole leaks in the top plate. It was attempted several times to seal the leaks with plastic and glue, and the filter was also lowered from its stand, in an effort to flood the top section with water and force all the air out. All tactics met with limited success. The final solution was to shut off the airlift that returned wastewater to the recirculation tank. Shutting the airlift off for a brief period of time (less than one minute) would allow the space between the upper plate and the top screen to become flooded with water, as the airlift was not siphoning water away from the filter. After allowing air flow back to the airlift, the samplers waited several minutes before obtaining the sample.

Results from the filter were divided into different data sets based on the $CBOD_5$ concentrations inside of the filter bed (low, mid, and high). The low organic substrate regime in this study described the condition in which the averaged filter bed $CBOD_5$ concentration was equal to or less than 10 mg/L. The mid-level substrate range indicates a filter bed $CBOD_5$ concentration between 10 and 30 mg/L. It should be noted that the data presented in Table 2 is for multiple passes through the filter bed, each pass lasting from 30 to 90 seconds.

Table 2 – Average Filter Results Under Different Filter Bed Concentration Levels

Experimental Series	$CBOD_5$			TSS		
	Total Load (kg/m^3.d)	Effluent (mg/L)	% Removal	Total Load (kg/m^3.d)	Effluent (mg/L)	% Removal
BF4 – Low Substrate	1.5	4.43	95.4	0.7	3.3	93.7
(n)	(18)	(18)	(18)	(7)	(7)	(7)
BF4 – Mid Substrate	2.6	14.0	88.2	1.6	14.3	80.1
(n)	(10)	(10)	(11)	(8)	(8)	(8)

The total loading rates in Table 2 were based on the entire system, the filter and the equalization basin, calculated from the following general equation:

$$Loading_{Total} = \frac{S * Q_{Total}}{V_{bed}}$$

Where S is the substrate, $CBOD_5$ or TSS, entering the filter bed, Q_{Total} is the flow rate through the entire system, and V_{bed} is the volume of the filter media. This loading rate should be differentiated from the bed loading, reported later in Table 4. The operational parameters for the same experimental regimes described above can be found in Table 3.

Table 3 – Operational Parameters For Different Substrate Regimes

Experimental Series	Filtration Rate (m/h)	Retention Time (min) One Pass	Retention Time (min) Total	Oxygen Uptake Rate ($kg/m^3.d$)
BF4 – Low Substrate	14.2	0.8	60.6	2.6
(n)	(19)	(19)	(19)	(19)
BF4 – Mid Substrate	13.0	0.9	39.4	1.5
(n)	(12)	(12)	(11)	(12)

Table 4 – Single Pass Results

Experimental Series	$CBOD_5$		TSS	
	Bed Loading ($kg/m^3.d$)	Removal %	Bed Loading ($kg/m^3.d$)	Removal %
BF4 – Low Substrate	6.4	21.1	4.2	12.7
(n)	(16)	(12)	(7)	(6)
BF4 – Mid Substrate	17.8	11.9	18.4	19.6
(n)	(11)	(11)	(8)	(8)

The bed loading rates in Table 4 were calculated from the following general equation

$$Loading = \frac{S * Q_{filter}}{V_{bed}}$$

Where S is the substrate, $CBOD_5$ or TSS, entering the filter bed, Q_{filter} is the flow rate through the filter, and V_{bed} is the volume of the filter media.

Discussion

The relationship between dissolved oxygen concentrations in the filter and $CBOD_5$ reduction was explored. Although a complex function of microbial population, loading to the filter, solids removal efficiency, and operational parameters, it was attempted to apply a generic, simplified relationship in order to evaluate overall performance at different bed concentrations. The strategy was to relate the dissolved oxygen consumed in the filter (OCF) by the bacteria to the removal rate of $CBOD_5$ (BOD_r) applied to the filter by a function labeled the MX Factor.
The OCF equation follows:

$$OCF = \frac{(DO_{in} - DO_{out})Q}{V_b}$$

Where DO_{in} and DO_{out} are the concentrations of DO entering and exiting the filter, respectively, Q is the flow rate through the filter, and V_b is the volume of beads (Malone and Beecher, 2000). The BOD_r equation is similar with $CBOD_5$ replacing DO in the above equation. The relationship between the two is expressed:

$$MX = \frac{BOD_r}{OCF}$$

It was assumed prior to this study that the minimum MX value would be one, however; this experimental effort has revealed that the minimum MX value is lower than one, and may be zero. This is best seen in periods of very low bed concentration (less than 8 mg/L $CBOD_5$), in which MX values are consistently less than one. It is believed that when MX values are less than one, some level of nitrification occurs.

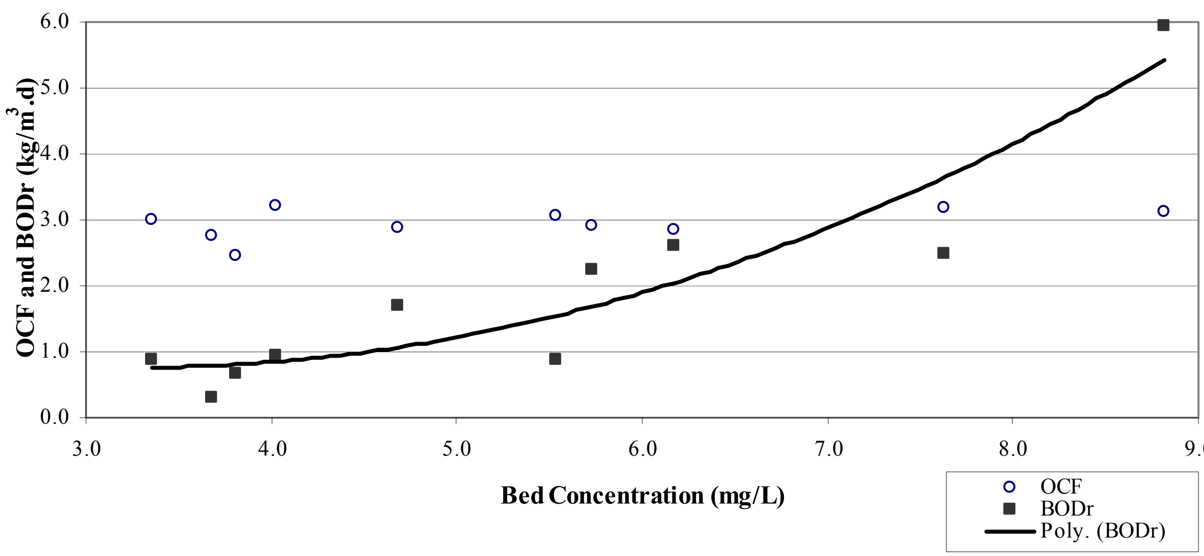


Figure 2. Nitrification occurred in the filter at low organic loadings.

The performance data obtained from Delta Prototype 4 was used to evaluate the relationship between the total BOD volumetric organic loading and the effluent quality. This information is useful in design considerations, and it provides a basis for comparison of this filter with other SLDM Filter configurations and other treatment technologies. The loading curve to the entire system of Delta Prototype 4 was developed and is shown below in Figure 3. The curve illustrates a range of organic loadings (i.e. $CBOD_5$ loadings) to the entire prototype (bead filter with multiple passes plus the equalization tank) per volume of media in the filter per day. A system loading of approximately 2.4 $kg/m^3.d$ would result in a $CBOD_5$ effluent concentration of 10 mg/L. Figure 4 is a similarly developed relationship, which illustrates the organic loadings to the filter alone on a single pass basis. A loading of approximately 12.7 $kg/m^3.d$ would result in a $CBOD_5$ effluent concentration of 10 mg/L. In comparison, Stensel et al. operated a full scale

Biological Aerated Filter (BAF) and was able to get a 10 mg/L BOD_5 effluent concentration for a organic (BOD_5) loading of approximately 1.4 to 1.8 kg/m^3.day (1988).

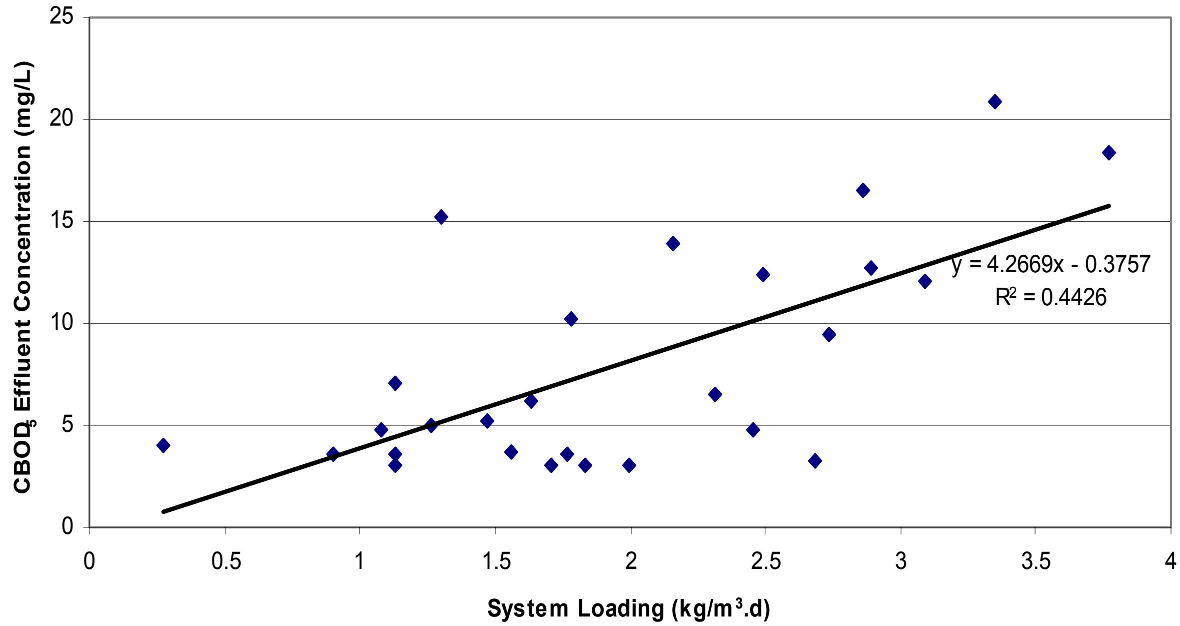


Figure 3. Organic System Loading Response Curve for Delta Prototype 4

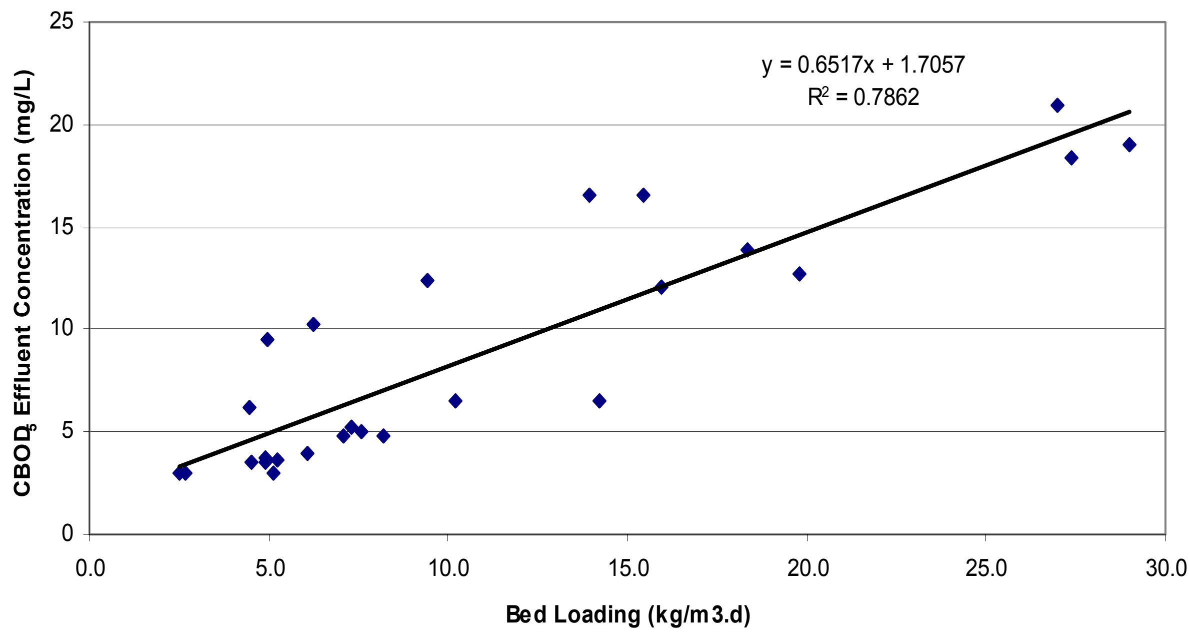
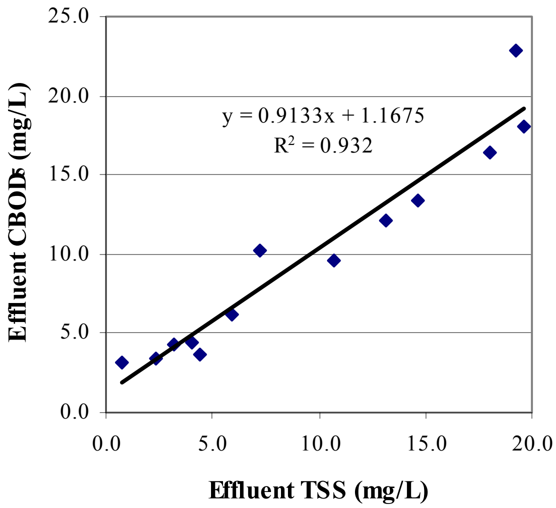


Figure 4. Organic Filter Loading Response Curve for Delta Prototype 4

The removal of suspended solids was an important consideration in these units when aiming to reduce total $CBOD_5$ concentrations for single and multi-pass regimes. The relationship between TSS and $CBOD_5$ in the effluent from the system reveals that a higher TSS removal would lower $CBOD_5$ concentrations. It was also observed that longer backwash intervals resulted in lower

TSS effluents. It is suspected that the higher TSS levels associated with short backwash intervals are an artifact of the configuration of the filter.



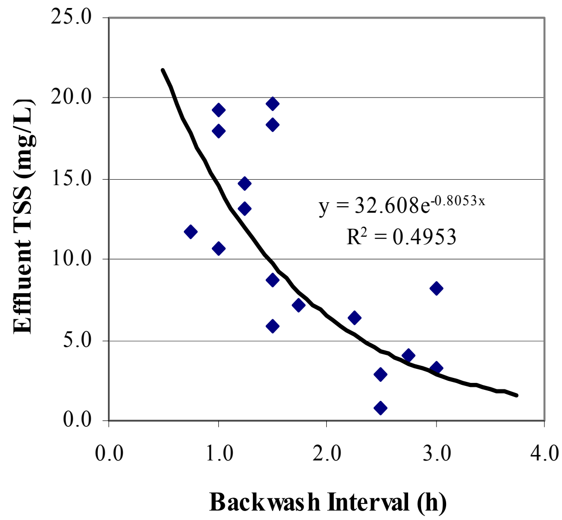


Figure 5a and 5b. TSS appears to contribute to the CBOD levels in the filter. Less frequent backwashing may help reduce the levels of both TSS and CBOD.

Conclusions

The consolidation of biological processes and physical operation into a single unit represents the ability of a bioclarifier to function as an entire secondary treatment train and, if protected from the natural oils and greases found in domestic wastewater, as a primary and secondary treatment system in a single unit. In addition, the SLDM Filter is independent of human interaction for operation and requires little maintenance. SLDM Filters could potentially be used as separate primary, secondary, or tertiary treatments, and possibly as a combined primary and secondary or secondary and tertiary treatment unit. If used as secondary treatment alone, a single SLDM Filter could be used to replace an activated sludge unit and secondary clarifier or a trickling filter and its associated secondary clarifier, substantially reducing the construction costs and the land requirement. A SLDM Filter could also be used in areas where traditional septic systems could be implemented.

Future Work

Delta Prototype 4 shed insight to the following issues, which should be appropriately addressed in future SLDM Filters:

- Both screens in the filter (below and above the bead bed) exhibited minimal head loss. This is of most significance to the lower filter and is thought to be attributed to the position of the backwashing air trigger relative to the lower screen.
- The relationships shown in Figure 5 suggest that increased solids removal would result in lower effluent CBOD concentrations. One means of control, which should be further evaluated, is the configuration of backwashing.
- The Delta 4 prototype exhibited MX values below one under periods of low organic loading, indicating the occurrence of nitrification. The experimental matrix of future prototypes could include ammonia analysis in order to determine the oxygen balance in SLDM Filters.

- The footprint of the SLDM filter may be decreased by an alternate design configuration.

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Appendix A: Prototype 4 Sampling Procedure Corrective Actions

Area of Concern	Previous Delta Prototype Procedure	Delta Prototype 4 Procedure
Glassware Handling	Glassware rinsed with tap water, followed by an abrupt rinse with Contrad Solution and another rinse with tap water.	Glassware rinsed with tap water and placed into a bath containing a solution of 2 to 5% Contrad. Glassware remained in solution for at least 2 hours, but usually 24 hours. After removing from Contrad bath, glassware was rinsed three times with "US Filter Water" (water passed through three resin columns for ion exchange) and then once with water from the Nanopure system in the WQL
Dissolved Oxygen Titration	Via pipet	Via buret
Dissolved Oxygen Reagent	No standardization	Standardization of Sodium Thisulfate performed
Total Suspended Solids Filter Preparation	Filters improperly prepared. Filters were not rinsed, but were placed in a 105 degree C oven prior to initial weight measurement.	Filter preparation according to Standard Methods: Filters rinsed with DI water and then put in 550 degree C muffle furnace for 20 minutes prior to initial weight measurement.
Total Suspended Solids Repeatability	TSS samples performed in duplicate.	TSS samples performed in triplicate
Total Suspended Solids Data Criteria	No criteria for blanks were observed.	Criteria for Blanks established to be ±3 mg/L.
Biochemical Oxygen Demand Repeatability	BOD samples performed in duplicate (at most).	BOD samples performed in triplicate.
Biochemical Oxygen Demand Data Criteria	No criteria for samples were observed.	Criteria from Standard Methods were observed. The blank drop was required to be ± 0.20 mg/L. The minimum dissolved oxygen drop after five days was required to be 2 mg/L. At least 1 mg/L was required to remain in the sample after five days.
Transport of Sample	Samples stored in an open topped plastic container, without ice for transport.	Samples stored on ice, in a closed ice chest during transport.

Final Report: BF6 Prototype

Experimental Report Number 2002.02

Cynthia Wagener

December 20, 2002

Louisiana State University

Institute for Ecological Infrastructure Engineering

Prototype 6 Experimental Report

Abstract

From November 2001 to May 2002, the Delta 6 Experimental Unit was in operation at the Delta Environmental Test Site, located in Denham Springs, Louisiana. During this period, the system was operated and evaluated at various hydraulic and organic loading rates, recirculation rates, backwash intervals, and ambient temperatures. Samples were subject to various analytical analyses, which were conducted in triplicate according to Standard Methods for the Examination of Water and Wastewater (APHA, 1995). Averaged results from this evaluation are presented.

Introduction

The fundamental design of most large scale domestic wastewater treatment plants is a classical unit operation configuration, consisting of primary clarification, biological treatment, and secondary clarification. This approach is effective, but results in treatment plants requiring a large footprint, high capital resources, and sophisticated staffs. Smaller communities may not have the resources for a large centralized plant and may instead opt for individual on-site treatment systems. Regardless of the means, these communities demand technologies that are robust and relatively simple to operate without any compromise of effluent quality. Such approaches can be developed systematically by consolidating the treatment train through utilization of technologies that can perform more than one process. A treatment system designed within these parameters could provide both large and small communities an attractive alternative to current systems.

Background

Static low-density media (SLDM) filters are known in the aquaculture community as Floating Bead Filters (FBFs). The units are currently widely employed as clarifiers or bioclarifiers in support of high-density recirculating production and holding systems for fish, reptiles, and crustaceans (Malone and Beecher, 2000; DeLosReyes and Lawson, 1996). Historically, SLDM filters have been used exclusively in aquaculture applications, therefore; most research has been in improving nitrification capacity, as that parameter has been identified as the limiting factor in bioclarifier performance for aquaculture applications (Sastry, 1999; Malone *et al*., 1993). SLDM Filter technology has not previously been applied to domestic wastewater treatment for concurrent biological and physical treatment. Technologies that have been employed in domestic wastewater treatment capacities similar to SLDM filters include: biological aerated filters (BAFs), trickling filters, and sand filters.

SLDM Filters are fixed-film filters, typically operated in an upflow configuration, in which a biofilm support medium is submerged in wastewater to create a large contact area for aerobic biological treatment. Air or oxygen is supplied to the filter without disturbing the media, resulting in a static bed and necessitating recirculation of the wastewater in order to maintain aerobic conditions. Airlift pumps can readily provide submerged packed beds challenged with elevated BOD levels the necessary high rate of recirculation and aeration demanded (Loyless and Malone 1998; Reinemann and Timmons, 1989). The combined floating media filter and airlift unit, when operated the packed bioclarification mode with a low headloss, results in an effective treatment strategy with the functions of concurrent recirculation, aeration, biofiltration, and secondary clarification all being effectuated by an air blower, recirculating tank, and a floating media filter.

The low-density plastic media acts as a carrier for biofilm and as a physical separation mechanism for solids. Heterotrophic bacteria attach themselves to the beads and utilize the organic matter in the waste stream as a carbon source for growth, while autotrophic, nitrifying bacteria convert ammonia to nitrate under conditions of low organic loading (Zhang et al, 1995). Concurrently, suspended solids in the waste stream are captured in the bed via surficial straining, deep bed filtration, and adsorption as the waste stream travels upward though the bead bed. The capture of solids in a SLDM Filter is known to be influenced by particle size, filter media size, flowrate, and solids accumulation (Ahmed, 1996).

The SLDM filter used in this study supplied oxygen through recirculation of the wastewater through the filter bed by means of airlift pumps. The wastewater made multiple passes through the filter bed with retention times of 30 seconds to one and a half minutes per pass. At the end of each pass, an airlift pump would return the wastewater to the equalization basin chamber. The airlift pump served the dual functions of water delivery and aeration.

The beds of SLDM filters are periodically expanded for removal of accumulated solids and excess biofilm (Malone and Beecher, 2000; Cooley, 1979). Backwashing or expansion of a bead bed can be accomplished by hydraulic, pneumatic, or mechanical means. SLDM Filters take advantage of the buoyancy of the media to minimize the water loss that would otherwise be associated with the high frequency washing needed to manage the biofilm. These units are capable of restricting water losses to periods of sludge removal, as opposed to other filters which can use ten percent of the product water (Stensel *et al.,* 1988) to twenty percent of the influent wastewater (M'Coy, 1997) to hydraulically wash the media. SLDM Filter units are virtually impervious to caking problems that can plague granular filters that are subject to high organic loads. Since backwash water loss is minimal in filters with internal propellers and in the drop filter units, the backwash frequency can be employed as a biofilm management tool (Malone *et al*. 1993). Additional biofilm management flexibility is obtained by altering the bead shape or by moving to a less aggressive washing format (Golz *et al*. 1999). However once the unit is selected, backwash frequency is the principle operation parameter used to enhance biofiltration performance.

Methods and Materials

The Delta 6 prototype filter was configured in the manner shown in Figure 1. The prototype received flow from a large, 1000 gallon tank that acted as both storage and as a primary clarifier. This was followed by the Delta 6 Experimental Unit, which consisted of a bead filter inside of a equalization/recirculation tank. The filter was two feet in diameter and contained 113.3L (4 ft^3) of biofilm carrier, with a bed depth of approximately 38 cm. The carrier was a modified media 3 to 5 mm in diameter, with a density of 0.90 kg/L, a porosity of 0.55, and with a total specific surface area of 1100 to 1250 m^2/m^3 (Malone *et al*., 1993). The Delta 6 Experimental Unit had a total capacity of 1.78 m^3 (four feet in diameter and five feet tall). The filter was placed in the middle of the tank, and a concentric partition, three feet in diameter, was also placed in the tank. The resulting effect was one tank with three separate chambers: an outside, atrium chamber; an inside chamber; and the filter bed. Airlift pumps were used to steady the direction of flow and circulate water from the filter to the inner chamber. In Figure 1, only one airlift is seen, although the system was equipped with two airlift pumps, which were located on a single axis that traversed through the center of the filter. Following the filter-tank combination was an effluent holding tank. This effluent sump contained a trash pump and was followed by a meter so that the total volume exiting through the system could be determined.

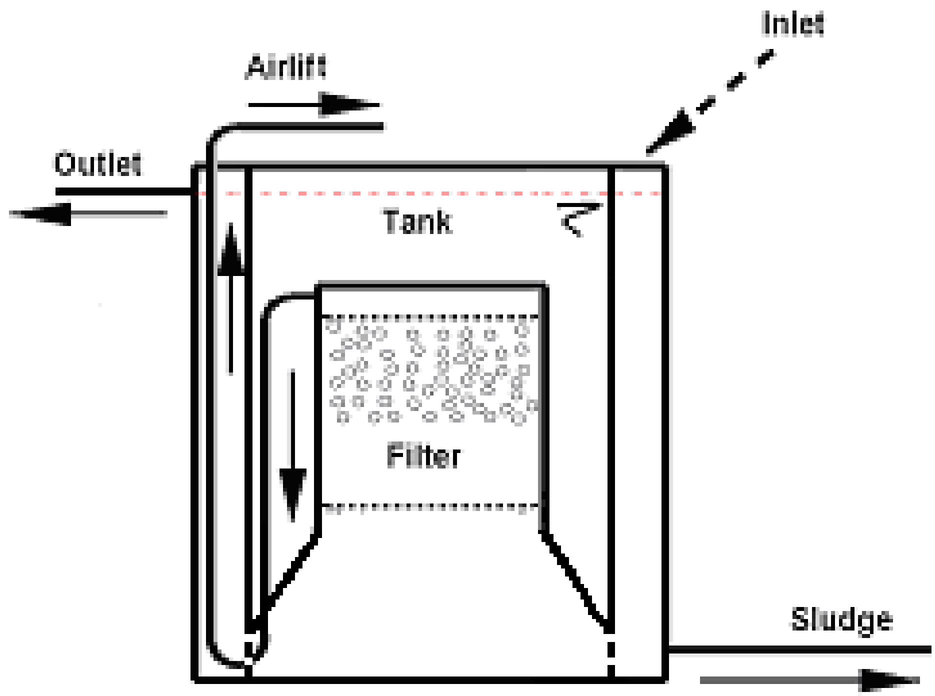

Figure 1. General Configuration of Delta Prototype 6.

The site was subject to highly variable flow characterized by morning and afternoon peaks. Temperature, pH, and flow measurements were recorded along with other operational parameters, such as backwash frequency, during each sampling event. The sampling procedures are outlined in the section below. Water quality parameters were tested in triplicate in the Water Quality Laboratory located at Louisiana State University, according to Standard Methods for the Examination of Water and Wastewater and include the following: $CBOD_5$ (5210B), DO (4500-O C), TSS (2540 D), and VSS (2540 E) (APHA, 1995).

Table 1. Influent Wastewater Characteristics

Parameter	Mean Value
$CBOD_5$, mg/L	146
(n)	(23)
TSS, mg/L	93
(n)	(20)
Temperature C	17.2
(n)	(28)

The entire system was operated for more than one month in an acclimation mode, prior to testing. During this period wastewater was circulated through the filter, but the backwashing frequency was lowered so that bacteria could populate the biofilm carrier.

Sampling Procedures

Upon arrival at the Delta Environmental Test site, the date and time was recorded and then a quick scan of the site commenced. Unlike previous prototypes, Delta Prototype 6 did not have a Backwash Indicator Line (BIL), so sampling commenced upon arrival. Samples interrupted by a backwashing event were considered not usable. If a backwash occurred while at the site, the time was recorded and the sampler would wait at least 45 minutes or until overflow from the filter resulted in effluent flow before sampling again; frequently the sampler would elect to return the next day to collect the sample. The amount of time it took for effluent flow to reappear was dependant on water usage, which was correlated with the time of day.

In situ measurements were then recorded. The number from the System Effluent Meter was recorded. This meter displayed the volume of effluent leaving the system. The temperature inside both chambers of the tank was also recorded with a glass mercury thermometer. The air flowrate (scfh) to the backwash air chamber was also recorded. Other observations concerning the site and the filter appearance were recorded.

The System Out sample (i.e. effluent from the entire system) was taken first. Typically two liters were taken for BOD and TSS analysis in a single two liter plastic bottle. If the filter had recently backwashed, this forced the sampler to wait until the filter produced overflow, the effluent.

The Recycle Out and Recycle In samples were taken next (flow out of and into the bead bed, respectively). Two liters of sample were taken in 2 L plastic bottles for BOD and TSS analysis, and three 300 mL BOD bottles were filled with each sample for DO analysis by Winkler Titration. An additional sample volume was taken (1 L) if a DO meter was used. The bottles for DO measurement were filled first. During this process, no air bubbles were allowed to travel through the sample lines, otherwise the sample was rejected and retaken. The BOD bottles were filled by inserting the tube into the bottle so that it touched the bottom of the bottle, and the bottles were allowed to overflow before the tubing was removed. After all the BOD bottles were filled, the two liter sample bottles were filled to capacity and allowed to overflow. After the samples were taken, the reagents used in the Azide modified Winkler DO procedure were added, in preparation for titration at the Water Quality Laboratory.

The recirculation flowrate was then taken via a bucket with a ten liter mark and a stopwatch. The bucket was allowed to sink as it filled to maintain a constant level of water in the tank. This measurement was taken three times. This measurement was not taken before any of the samples were taken, in an effort to prevent any disturbance to the samples. Delta Prototype 6 was equipped with two airlifts, positioned 180 degrees apart on an axis that traversed through the center of the filter. Flow measurements were taken for each airlift, and then added together for the total recirculation flow.

In January 2002, a backmix hole was drilled through the concentric inner ring, which separated the inner chamber from the outer atrium. This hole was made in an effort to return some of the low strength wastewater to dilute the higher strength wastewater in the outside chamber, and as a means to provide additional dissolved oxygen to the outside chamber. Inside of the hole was a short piece of PVC pipe that carried water leaving the airlift to the outer chamber. The backmixing rate was controlled by two ball valves, one ball valve controlled the amount of water leaving the airlift, and the other valve opened or shut the backmixing hole. After the backmix hole was drilled, two sets of recirculation flow rates were measured. First the recirculation flow rate with the backmix flow was measured, and then the recirculation flow with the backmix on-off valve in the off position was measured. After this measurement was taken, the on-off valve was once again put into the on position. With these two measurements, the backmix flow rate could be determined. If a backwash occurred after this point, the samples were considered still useable, however, if a backwash occurred anytime between collection of the System Out sample and this point, the samples were be discarded and these steps were repeated once there was overflow into the final sump.

The headloss through the filter was then measured with a ruler in units of centimeters. In Prototype 6, only the total headloss was measured, as the evaluation of a previous prototype revealed that the screen separating the bead bed and the lower part of the filter was not subject to clogging when configured in the manner used in this design (Wagener, 2002). If a backwash occurred after the recirculation flowrate was measured, the head loss was not recorded, as that measurement would not accurately reflect the headloss prior to backwashing.

The System In sample was then taken in a 1 L plastic bottle. The first 100 to 400 milliliters were discarded, as it contained large solids that settled into the fitting and did not accurately represent the sample. Then the bottle was filled to near capacity. The samples were placed into an ice chest and put on ice for transport back to the Louisiana State University Water Quality Laboratory. Samples typically reached the laboratory within one hour of collection.

The pH of the samples was then checked with a portable pH meter, if it was brought to the site. The pH was checked in the sample bottles moving from least concentration to most and the probe was washed between bottles with DI water.

Sludge was discarded at random intervals until the effluent was no longer dark (approximately five to ten gallons). Sludge disposal was recorded when it occurred.

After sampling ended, the sampler then made additional observations around the site where the Delta 6 prototype equipment was located and checked to verify that everything looked normal and was operating functionally. If the Backwash Frequency needed alteration, it was done at this point, and the new air flow rate was recorded. Finally, the sampler was to make sure that the cover for the recirculating tank was on so that rain and sunlight was blocked. This was part of the continuous effort to prevent algae from growing in the recirculating tank

Results

Experimental results for the Delta Prototype 6 evaluation period from December 2001 to May 2002 have shown carbonaceous biochemical oxygen demand ($CBOD_5$) concentrations to decrease from 146 mg/L to 25 mg/L on average for this period and through multiple passes of the filter. Total suspended solids (TSS) concentrations have been shown to decrease from 93 mg/L to 33 mg/L on the average. During the sampling period, field visits were made on 46 days, of which field sampling for at least one analytical parameter occurred 24 times. The entire collected data set can be found in Appendix A. It should be noted that not every data set in

Appendix A is complete or useable; this is attributed to either issues in the field or a failure to meet criteria in laboratory analysis.

The most notable field issue was air intrusion into the filter. Air was collecting above the top plate of the filter, which resulted in air bubbles in the “Recycle Out” sample line. The problem was present with varying degrees of severity throughout the entire operational period. One possible source was a pinhole leak from the backwash air chamber, from which air was collecting above the top plate and was not being completely forced out. The problem was compensated for by allowing the sample lines to run until no air bubbles appeared. If the sampling lines continued to have bubbles after running for approximately five minutes, tap water was run into the sampling line in an effort to flood the top section, and the samplers waited at least fifteen minutes prior to collecting samples. This procedure was avoided whenever possible.

Results from the filter were divided into different data sets based on the $CBOD_5$ concentrations inside of the filter bed (low, mid, and high). The low organic substrate regime in this study described the condition in which the averaged filter bed $CBOD_5$ concentration was equal to or less than 10 mg/L. The mid- and high level substrate range indicates a filter bed $CBOD_5$ concentration between 10 and 30 mg/L, and over 30 mg/L, respectively. It should be noted that the data presented in Table 2 is for multiple passes through the filter bed, each pass lasting from 30 to 90 seconds.

Table 2 – Average Filter Results Under Different Filter Bed Concentration Levels

Experimental Series	$CBOD_5$			TSS		
	Total Load ($kg/m^3.d$)	Effluent (mg/L)	% Removal	Total Load ($kg/m^3.d$)	Effluent (mg/L)	% Removal
BF6 – Low Substrate	1.3	4.1	97.1	0.6	6.7	90.2
(n)	(3)	(3)	(3)	(3)	(3)	(3)
BF6 – Mid Substrate	2.0	14.1	90.0	1.3	22.1	74.7
(n)	(10)	(10)	(10)	(10)	(10)	(10)
BF6 – High Substrate	2.9	49.8	69.5	2.1	52.3	50.0
(n)	(5)	(5)	(5)	(8)	(8)	(8)

The total loading rates in Table 2 were based on the entire system, the filter and the equalization basin, calculated from the following general equation:

$$Loading_{Total} = \frac{S * Q_{Total}}{V_{bed}}$$

Where S is the substrate, $CBOD_5$ or TSS, entering the filter bed, Q_{Total} is the flow rate through the entire system, and V_{bed} is the volume of the filter media. This loading rate should be differentiated from the bed loading, reported later in Table 4. The operational parameters for the same experimental regimes described above can be found in Table 3.

Table 3 – Operational Parameters for Different Substrate Regimes

Experimental Series	Filtration Rate (m/h)	Retention Time (min) One Pass	Retention Time (min) Total	Oxygen Uptake Rate (kg/m^3.d)
BF6 – Low Substrate	13.2	1.0	98.3	1.6
(n)	(4)	(4)	(4)	(4)
BF6 – Mid Substrate	13.1	1.0	62.5	1.3
(n)	(10)	(10)	(10)	(10)
BF6 – High Substrate*	10.8	1.3	40.0	0.6
(n)	(10)	(10)	(10)	(10)

*High Substrate samples were consistently oxygen limited. DO concentrations in the filter bed averaged less than 1 mg/L.

Table 4 – Single Pass Results

Experimental Series	$CBOD_5$		TSS	
	Bed Loading (kg/m^3.d)	Removal %	Bed Loading (kg/m^3.d)	Removal %
BF6 – Low Substrate	5.3	29.5	8.4	22.9
(n)	(2)	(2)	(3)	(3)
BF6 – Mid Substrate	14.8	16.0	23.4	18.2
(n)	(9)	(9)	(10)	(10)
BF6 – High Substrate	37.7	14.2	35.7	13.8
(n)	(5)	(4)	(8)	(8)

The bed loading rates in Table 4 were calculated from the following general equation

$$Loading = \frac{S * Q_{filter}}{V_{bed}}$$

Where S is the substrate, $CBOD_5$ or TSS, entering the filter bed, Q_{filter} is the flow rate through the filter, and V_{bed} is the volume of the filter media.

Discussion

The performance data obtained from Delta Prototype 6 was used to evaluate the relationship between the total BOD volumetric organic loading and the effluent quality. This information is useful in design considerations, and it provides a basis for comparison of this filter with other SLDM Filter configurations and other treatment technologies. The loading curve to the entire system of Delta Prototype 6 was developed and is shown below in Figure 2. The curve illustrates a range of organic loadings (i.e. $CBOD_5$ loadings) to the entire prototype (bead filter with multiple passes plus the equalization tank) per volume of media in the filter per day. A system loading of approximately 1.2 kg/m^3.d would result in a $CBOD_5$ effluent concentration of

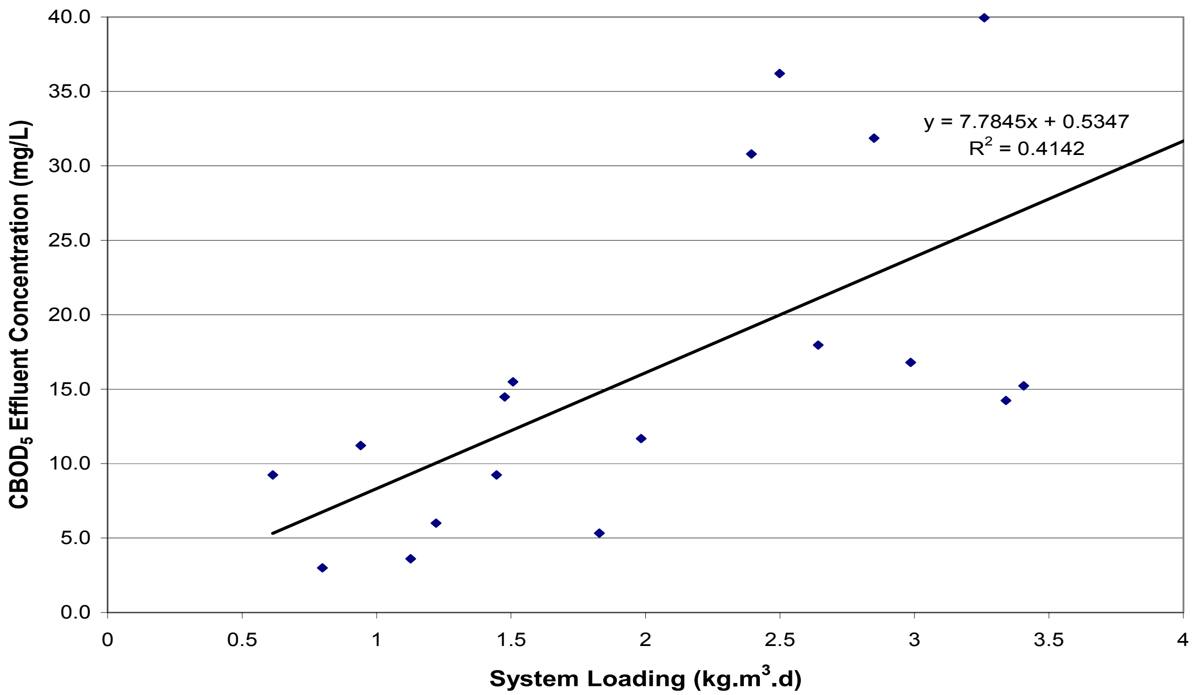

Figure 2. Organic System Loading Response Curve for Delta Prototype 6

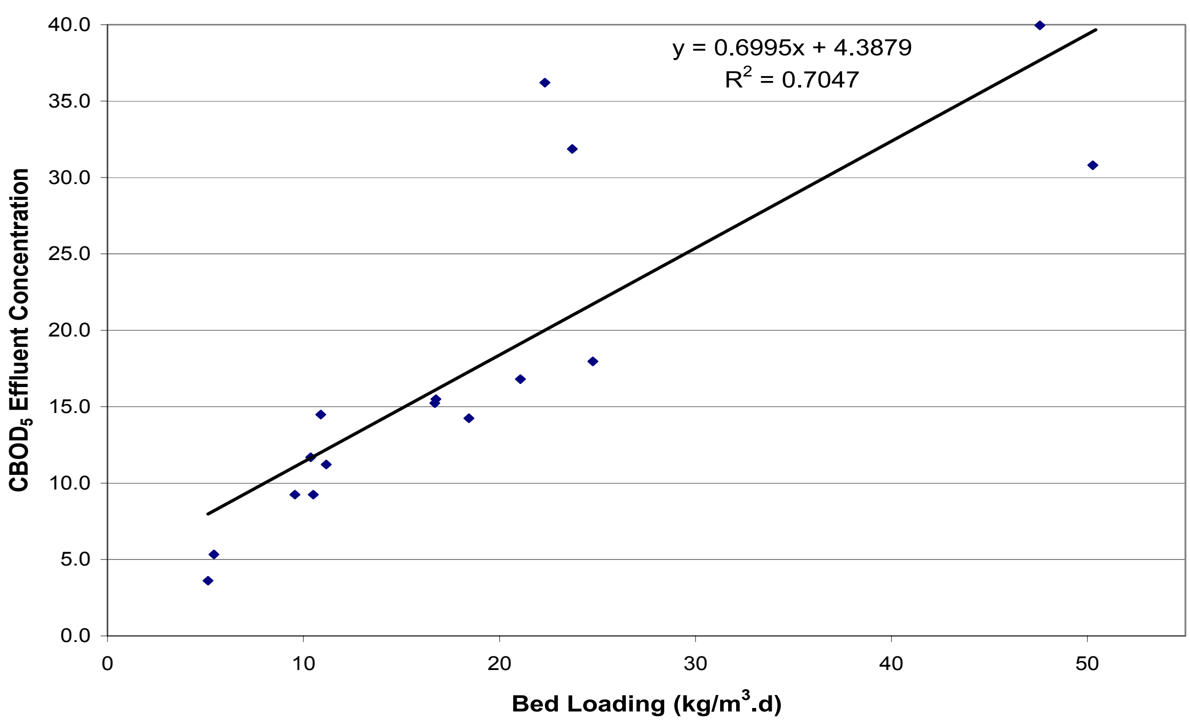

Figure 3. Organic Filter Loading Response Curve for Delta Prototype 6

10 mg/L. Figure 3 is a similarly developed relationship, which illustrates the organic loadings to the filter alone on a single pass basis. A loading of approximately 8.0 kg/m^3.d would result in a $CBOD_5$ effluent concentration of 10 mg/L. In comparison, Stensel et al. operated a full scale

Biological Aerated Filter (BAF) and was able to get a 10 mg/L BOD_5 effluent concentration for an organic (BOD_5) loading of approximately 1.4 to 1.8 kg/m^3.day (1988).

In a previous SLDM Filter Prototype, a strong correlation between effluent solids and effluent biochemical oxygen demand was shown (Wagener, 2002). From this relationship, it was suggested that more effective CBOD removal would be achieved by an increased removal of TSS. A similar pattern was observed in Delta Prototype 6. This issue was further evaluated by analyzing the influent and effluent for both soluble and total $CBOD_5$. It was found that while the majority of the influent was associated with soluble $CBOD_5$ (67%), $CBOD_5$ in the effluent was dominated by particulate matter (the soluble $CBOD_5$ was found to be 29%).

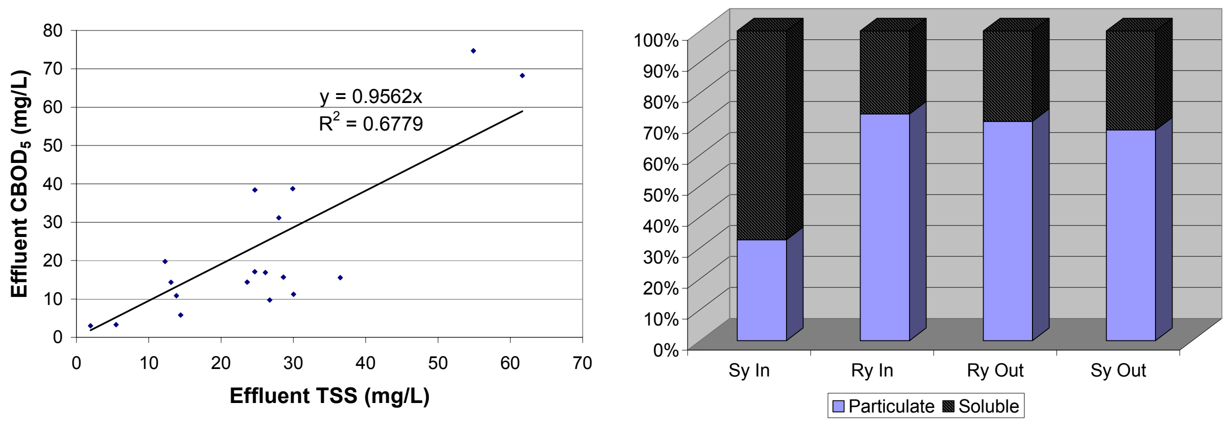

Figure 4a and 4b. More effective removal of solids would result in lower organic levels in the effluent waste stream.

Conclusions

The consolidation of biological processes and physical operation into a single unit represents the ability of a bioclarifier to function as an entire secondary treatment train and, if protected from the natural oils and greases found in domestic wastewater, as a primary and secondary treatment system in a single unit. In addition, the SLDM Filter is independent of human interaction for operation and requires little maintenance. SLDM Filters could potentially be used as separate primary, secondary, or tertiary treatments, and possibly as a combined primary and secondary or secondary and tertiary treatment unit. If used as secondary treatment alone, a single SLDM Filter could be used to replace an activated sludge unit and secondary clarifier or a trickling filter and its associated secondary clarifier, substantially reducing the construction costs and the land requirement. A SLDM Filter could also be used in areas where traditional septic systems could be implemented.

Future Work

Delta Experimental Unit 6 further illustrated the need for a more optimized backwashing regime. An alteration to the prototype design in which the compartments are more separated with more controlled flow and also in which the backwash is more controlled may yield a better loading response curve. Solids capture was a key point of concern in this prototype.

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APPENDIX B: BATCH STUDY DATA

01 August 2001 Batch Study on BF3

Date	day time		1-Aug-2001 0	1-Aug-2001 1	1-Aug-2001 2	1-Aug-2001 3	1-Aug-2001 4	1-Aug-2001 5
Sampler			CW	CW	CW	CW	CW	CW
BW Interval	hr		2.00	2.00	2.00	2.00	2.00	2.00
Temp	C		33.0	34.5	35.0	40.0	41.5	42.0
$CBOD_5$	Analyst		CW, QW	CW, QW	CW, QW	CW, QW	CW, QW	CW, QW
Ry In	mg/L		< 88	23.60	20.63	15.30	15.80	15.29
Ry Out	mg/L		57.28	22.57	16.13	13.7 < x < 15	> 14.8	> 12
DO								
DO In	mg/L		1.60	1.77	2.17	2.27	2.93	2.83
DO Out	mg/L		0.33	0.58	0.63	0.93	1.07	1.37
TSS	Analyst		CW	CW	CW	CW	CW	CW
Ry In	mg/L		39.56	14.22	13.67	9.00	13.60	11.11
Ry Out	mg/L		33.78	13.78	12.67	10.13	12.27	10.44

Note: All results from analytical tests represent averaged results.
$CBOD_5$ analysis was perfomed in duplicate with three dilutions
DO analysis was via Winkler Method and performed in triplicate
TSS analysis was performed in tripliate

13 August 2001 Batch Study on BF3

Date	day time		13-Aug-2001 0	13-Aug-2001 1	13-Aug-2001 2	13-Aug-2001 3	13-Aug-2001 4	13-Aug-2001 5
Sampler			CW,QW	CW,QW	CW,QW	CW,QW	CW,QW	CW,QW
Flow	l/min		2.22	3.00	2.61	2.50	2.31	2.14
Qr	l/min		4.62	10.00	6.67	8.57	5.45	8.57
BW Interval	hr		2.0	2.0	2.0	2.0	2.0	2.0
Temp	C		31.0	34.5	36.5	37.5	38.0	38.5
$CBOD_5$	Analyst		CW,QW	CW,QW	CW,QW	CW,QW	CW,QW	CW,QW
Ry In	mg/L		< 30	< 17	< 13.4	< 8	9.00	7.05
Ry Out	mg/L		23.95	< 13.4	< 12.1	8.19	8.01	6.20
DO								
DO In	mg/L		4.28	4.05	4.58	4.45	4.93	4.85
DO Out	mg/L		1.00	0.98	1.20	1.20	1.05	1.45
OUR	$kg/m^3.d$		1.54	3.12	2.29	2.83	2.15	2.96
BODr	$kg/m^3.d$						0.55	0.74
MX							0.26	0.25
BOD Bed	mg/L						8.50	6.63
Bed Load	$kg/m^3.d$						4.99	6.15
TSS	Analyst		CW	CW	CW	CW	CW	CW
Ry In	mg/L		24.81	10.22	7.47	5.97	8.11	4.87
Ry Out	mg/L		12.42	7.19	6.67	4.89	6.42	3.93
VSS	Analyst		CW	CW	CW	CW	CW	CW
Ry In	mg/L		21.11	10.22	7.07	5.47	7.33	4.73
Ry Out	mg/L		11.21	6.84	4.80	4.67	6.42	3.87

Note: All results from analytical tests represent averaged results.
$CBOD_5$ analysis was perfomed in duplicate with three dilutions
DO analysis was via Winkler Method and performed in triplicate
TSS analysis was performed in tripliate

20 August 2001 Batch Study on BF3

Date	day time		20-Aug-2001 0	20-Aug-2001 1	20-Aug-2001 2	20-Aug-2001 3	20-Aug-2001 4	20-Aug-2001 5
Sampler			CW,QW	CW,QW	CW,QW	CW,QW	CW,QW	CW,QW
Flow	l/min		3.10	2.10	2.63	1.35	1.14	1.01
Recycle Flow	l/min		6.99	9.32	9.45	8.11	9.53	10.72
BW Interval	hr		2.5	2.5	2.5	2.5	2.5	2.5
Temp	C		32.5	37.0	39.0	40.5	40.0	40.0
$CBOD_5$	Analyst		CW,QW	CW,QW	CW,QW	CW,QW	CW,QW	CW,QW
Ry In	mg/L		60.87	25.54	22.03	22.95	16.04	14.17
Ry Out	mg/L		42.55	23.37	19.56	20.01	15.80	13.13
DO								
DO In	mg/L		3.67	3.33	3.37	4.07	3.92	3.98
DO Out	mg/L		0.35	0.33	0.37	0.75	0.83	1.27
OUR	$kg/m^3.d$		2.36	2.84	2.88	2.73	2.99	2.96
BODr	$kg/m^3.d$		13.01	2.06	2.37	*2.42*	0.23	1.13
MX			5.52	0.72	0.82	0.89	0.08	0.38
BOD Bed	mg/L		51.71	24.45	20.79	21.48	15.92	13.65
Bed Load	$kg/m^3.d$		43.24	24.20	21.17	18.92	15.54	15.44
TSS	Analyst		CW,QW	CW,QW	CW,QW	CW,QW	CW,QW	CW,QW
Ry In	mg/L		46.33	20.89	17.73	23.60	14.53	10.13
Ry Out	mg/L		28.00	20.89	16.67	19.73	12.40	8.44
VSS	Analyst		CW,QW	CW,QW	CW,QW	CW,QW	CW,QW	CW,QW
Ry In	mg/L		43.33	19.11	16.13	21.73	14.00	10.00
Ry Out	mg/L		26.67	18.89	15.47	18.11	11.47	7.78

Note: All results from analytical tests represent averaged results.
$CBOD_5$ analysis was perfomed in duplicate with three dilutions
DO analysis was via Winkler Method and performed in triplicate
TSS analysis was performed in tripliate

APPENDIX C: BF4 DATA

Legend for Use in Appendix C

Solid highlights indicate the sample was experiencing a chemical interference that manifested itself in the Winkler titration procedure. Result of the chemical interference was the release of a pinkish-brownish gas that was released in the headspace of the BOD bottles containing DO samples after preservation. This gas was suspected to be iodine, released from the alkalai-iodide-azide solution.

= chemical interference

During the time of chemical interference (July and August 2001), YSI dissolved oxygen probes were used to determine the DO in the samples. This is indicated by use of italics.

italics = DO probe measurement

Analytical samples for DO, $CBOD_5$, TSS, and VSS were performed in triplicate, and in the case of $CBOD_5$, with two dilutions for all samples other than the system influent. Hach BOD QC samples were regularly analyzed for the $CBOD_5$ samples and standardization of the sodium thiosulfate titrant for Winkler's testing was consistently practiced. DO probes were regularly checked against Winkler titration. TSS and VSS samples were always accompanied with a blank, which was tested in triplicate along with the samples, as were the $CBOD_5$ samples.

~~number~~ = error with data

On a few occasions (such as October 31), the $CBOD_5$ samples were read at an incorrect time or the blank values were too high. The values obtained are shown, although they were not used in data analysis. Precise details explaining these data points are embedded as comments in the spreadsheet files on the companion CD for this document.

Comment on Sys Meter

The values given in this appendix are the total flow values recorded from the site visits. These numbers were processed to determine the daily flow by assuming that flow was generated for ten hours per day at the facility and that no flow was generated during weekends and over holidays when the facility was empty. These values may also be found on the companion CD.

BF4 July 2001 Data

Date	day		3-Jul-2001	6-Jul-2001	9-Jul-2001	12-Jul-2001	13-Jul-2001	16-Jul-2001
	time		3:00 PM	1:30 PM	2:30 PM	3:30 PM	11:00 AM	2:30 PM
Sampler			CW & QW	CW & QW	CW & QW	CW & QW	CW & QW	QW
Qr	m^3/d		79.09	88.02	89.01	85.94		78.22
Sys Meter	gal		10575.60	10780.20	10894.00	11239.20	11393.60	11840.20
BW Interval	hr			3.00	4.00	3.00	3.00	3.00
Temp	C		31.50	30.50	34.20	32.50	29.50	31.50
$CBOD_5$	Analyst							
Sy In	mg/L							
Ry In	mg/L						12.11	
Ry Out	mg/L						< 7.5	
Sy Out	mg/L							
DO	Analyst		CW & QW	CW & QW	CW & QW	CW & QW	CW & QW	QW
DO In	mg/L		6.20	5.36	16.93	1.67	8.23	11.33
DO Out	mg/L		5.88	4.63	15.80	1.05	3.93	7.20
OUR	$kg/m^3.d$		0.59	1.52	2.37	1.25		7.61
BODr	$kg/m^3.d$							
MX								
BOD Bed	mg/L							
Bed Load	$kg/m^3.d$							
TSS	Analyst			CW				
Sy In	mg/L			52.59				
Ry In	mg/L			8.27				
Ry Out	mg/L			8.00				
Sy Out	mg/L			8.27				
VSS	Analyst			CW				
Sy In	mg/L			44.81				
Ry In	mg/L			7.07				
Ry Out	mg/L			6.53				
Sy Out	mg/L			6.40				
Head Loss								
Top	cm							
Bed	cm							
Bottom	cm							
Total	cm							
pH								
Sy In				6.96	7.04	7.02		7.05
Ry In				7.10	7.15	7.10		7.12
Ry Out				7.29	7.33	7.26		7.32
Sy Out				7.23	7.25	7.19		7.20
Filtered $CBOD_5$								
Ry In	mg/L							
Ry Out	mg/L							

BF4 July 2001 Data

Date	day / time		17-Jul-2001 1:00 PM	18-Jul-2001 3:30 PM	19-Jul-2001 4:15 PM	23-Jul-2001 2:45 PM	27-Jul-2001 12:00 PM	31-Jul-2001 3:45 PM
Sampler			QW	CW & QW	CW	QW	CW	QW
Qr	m^3/d		70.49	72.44		62.87	42.35	
Sys Meter	gal		12060.40	12130.60	12300.00	12870.30		
BW Interval	hr		3.00	3.25		na		
Temp	C		30.50	31.50	32.00	32.70	31.30	
$CBOD_5$	Analyst				CW		CW	
Sy In	mg/L							
Ry In	mg/L				9.25		29.10	
Ry Out	mg/L				7.63		18.98	
Sy Out	mg/L							
DO	Analyst		QW	CW	CW	QW	CW	
DO In	mg/L		12.37	16.65	*3.27*	8.57	0.96	
DO Out	mg/L		8.77	16.30	*1.50*	7.32	0.40	
OUR	$kg/m^3.d$		5.97	0.60		1.85	0.56	
BODr	$kg/m^3.d$						10.08	
MX							18.07	
BOD Bed	mg/L				8.44		24.04	
Bed Load	$kg/m^3.d$						29.00	
TSS	Analyst							
Sy In	mg/L							
Ry In	mg/L							
Ry Out	mg/L							
Sy Out	mg/L							
VSS	Analyst							
Sy In	mg/L							
Ry In	mg/L							
Ry Out	mg/L							
Sy Out	mg/L							
Head Loss								
Top	cm							
Bed	cm							
Bottom	cm							1.00
Total	cm							13.00
pH								
Sy In								
Ry In				6.89				
Ry Out				6.96				
Sy Out								
Filtered $CBOD_5$								
Ry In	mg/L						14<x<15	
Ry Out	mg/L						9.38	

BF4 August 2001 Data

Date	day		1-Aug-2001	2-Aug-2001	9-Aug-2001	10-Aug-2001	13-Aug-2001
	time		12:00 PM	through	12:30 PM	3:00 PM	3:30 PM
Sampler			CW & QW	8-Aug-2001	CW & QW	CW & QW	CW & QW
Flow	m^3/d		22.70	corrected	39.87	42.28	50.85
Sys Meter	gal		15100.80	leaking	16220.40	16260.20	16530.10
BW Interval	hr			from top	0.50	1.00	
Temp	C		31.60	plate	32.80	30.80	28.20
$CBOD_5$	Analyst						
Sy In	mg/L						
Ry In	mg/L						
Ry Out	mg/L						
Sy Out	mg/L						
DO	Analyst				QW,CW	QW,CW	CW
DO In	mg/L				5.37	*6.00*	*5.28*
DO Out	mg/L				2.67		*2.85*
OUR	$kg/m^3.d$				2.53		*2.91*
BODr	$kg/m^3.d$						
MX							
BOD Bed	mg/L						
Bed Load	$kg/m^3.d$						
TSS	Analyst						
Sy In	mg/L						
Ry In	mg/L						
Ry Out	mg/L						
Sy Out	mg/L						
VSS	Analyst						
Sy In	mg/L						
Ry In	mg/L						
Ry Out	mg/L						
Sy Out	mg/L						
Head Loss							
Top	cm						
Bed	cm						
Bottom	cm		1.0		1.0		0.5
Total	cm		14.0		13.0		19.0
pH							
Sy In							
Ry In							
Ry Out							
Sy Out							

BF4 August 2001 Data

Date	day		14-Aug-2001	16-Aug-2001	17-Aug-2001	21-Aug-2001	24-Aug-2001
	time		4:00 PM	12:00 PM	11:30 AM	12:00 PM	2:00 PM
Sampler			CW	CW & QW	CW & QW	CW & QW	QW
Flow	m^3/d		54.30	52.40	60.95	59.63	57.01
Sys Meter	gal		16670.30	16926.20	16964.10	17272.80	17790.20
BW Interval	hr			3.00		2.25	2.25
Temp	C		28.30			30.60	30.30
$CBOD_5$	Analyst		CW	CW & QW	CW & QW	QW	CW & QW
Sy In	mg/L		> 118	88.68	76.05	84.05	
Ry In	mg/L		11.14	6.15	4.24	3.67	
Ry Out	mg/L		6.49	< 4.8	< 4.8	3.04	
Sy Out	mg/L		< 4.8		< 4.0		
DO	Analyst		CW	QW,CW	QW,CW	QW,CW	QW
DO In	mg/L		*5.50*	*6.05*	*6.15*	*5.38*	*5.50*
DO Out	mg/L		*3.05*	*4.41*	*4.36*	*3.25*	*3.24*
OUR	$kg/m^3.d$		*3.13*	*2.02*	*2.57*	*2.99*	*3.03*
BODr	$kg/m^3.d$		5.94			0.88	
MX			1.90			0.29	
BOD Bed	mg/L		8.81			3.35	
Bed Load	$kg/m^3.d$		14.23	7.59	6.08	5.14	
TSS	Analyst						
Sy In	mg/L						
Ry In	mg/L						
Ry Out	mg/L						
Sy Out	mg/L						
VSS	Analyst						
Sy In	mg/L						
Ry In	mg/L						
Ry Out	mg/L						
Sy Out	mg/L						
Head Loss							
Top	cm						
Bed	cm						
Bottom	cm			1.0	0.5		0.5
Total	cm			14.0	14.0		15.0
pH							
Sy In							
Ry In							
Ry Out							
Sy Out							

BF4 September 2001 Data

Date	day		7-Sep-2001	12-Sep-2001	14-Sep-2001	18-Sep-2001	19-Sep-2001
	time		3:30 PM	1:00 PM	1:00 PM	11:00 AM	11:45 PM
Sampler			CW, QW, SB	CW,QW,SB	CW,QW,SB	DJ,QW	CW
Qr	m^3/d		30.07	50.06	52.60	53.67	50.92
Sys Meter	gal		18914.80	19446.50	19792.90	20394.50	20599.30
BW Interval	hr		2.75	3.00	3.00	3.00	6.00
Temp	C		30.30	31.50	29.50	30.50	29.00
$CBOD_5$	Analyst		CW,SB	CW,SB	CW,QW	CW,QW	CW,QW,SB
Sy In	mg/L		> 131	139.20	95.44	90.00	104.07
Ry In	mg/L		19.71	8.69	5.89	< 4	4.08
Ry Out	mg/L		16.56	6.57	5.19	3.24	3.53
Sy Out	mg/L		12.05	6.52	6.05	3.89	3.65
DO	Analyst		CW	CW	SB	CW	QW
DO In	mg/L		*3.61*	3.57	*3.02*	*3.65*	*3.63*
DO Out	mg/L		*0.94*	0.87	*0.55*	*1.52*	*1.58*
OUR	$kg/m^3.d$		*1.89*	*3.18*	*3.05*	*2.69*	*2.46*
BODr	$kg/m^3.d$		2.23	*2.50*	0.87		0.66
MX			1.18	0.79	0.28		0.27
BOD Bed	mg/L		18.13	7.63	5.54		3.81
Bed Load	$kg/m^3.d$		13.95	10.23	7.29		4.89
TSS	Analyst						CW,QW,SB
Sy In	mg/L						60.00
Ry In	mg/L						4.00
Ry Out	mg/L						no sample
Sy Out	mg/L						4.44
VSS	Analyst						
Sy In	mg/L						55.33
Ry In	mg/L						4.00
Ry Out	mg/L						no sample
Sy Out	mg/L						4.00
Head Loss							
Top	cm						
Bed	cm						
Bottom	cm			1.0	0.5	0.5	1.0
Total	cm			16.5	15.0	15.0	15.0
pH							
Sy In							
Ry In							
Ry Out							
Sy Out							

BF4 September 2001 Data

Date	day		20-Sep-2001	25-Sep-2001	26-Sep-2001	27-Sep-2001
	time		1:00 PM	11:30 AM	3:00 PM	11:00 AM
Sampler			SB,QW	QW	CW	SB
Qr	m^3/d		50.47	52.29	50.49	57.31
Sys Meter	gal		20753.00	21709.00	21852.60	22067.90
BW Interval	hr		2.75	2.50		2.50
Temp	C			24.10	23.80	22.70
$CBOD_5$	Analyst		CW	QW	CW	QW,SB
Sy In	mg/L		93.19	82.00	95.40	
Ry In	mg/L		3.81	6.64	4.42	<3
Ry Out	mg/L		3.56	4.83	3.62	<3
Sy Out	mg/L		4.41	4.59	3.44	< 3
DO	Analyst		QW	CW	CW	SB
DO In	mg/L		*4.05*	*3.90*	*4.05*	*5.15*
DO Out	mg/L		*1.72*	*1.53*	*1.35*	*2.83*
OUR	$kg/m^3.d$		*2.77*	*2.91*	*3.21*	*3.12*
BODr	$kg/m^3.d$		0.30	*2.23*	0.95	
MX			0.11	0.76	0.30	
BOD Bed	mg/L		3.69	5.74	4.02	
Bed Load	$kg/m^3.d$		4.53	8.17	5.25	
TSS	Analyst		CW,QW,SB		CW, MB	CW, MB
Sy In	mg/L		65.33		70.00	no sample
Ry In	mg/L		5.33		4.22	2.67
Ry Out	mg/L		4.33		3.33	2.22
Sy Out	mg/L		4.00		2.33	2.89
VSS	Analyst					
Sy In	mg/L		62.67		68.00	no sample
Ry In	mg/L		4.33		4.00	2.67
Ry Out	mg/L		3.78		3.22	2.11
Sy Out	mg/L		3.56		2.11	2.67
Head Loss						
Top	cm					
Bed	cm					
Bottom	cm		0.50		1.0	1.0
Total	cm		15.00		12.0	12.5
pH						
Sy In						
Ry In						
Ry Out						
Sy Out						

BF4 October 2001 Data

Date	day		2-Oct-2001	3-Oct-2001	4-Oct-2001	8-Oct-2001	9-Oct-2001	22-Oct-2001
	time		11:00 AM	4:00 PM	1:00 PM	3:00 PM	2:00 PM	10:30 AM
Sampler			QW	CW	SB	SB	QW	QW
Qr	m^3/d		44.10	36.83	34.33	35.57	39.86	33.48
Sys Meter	gal		229045.00	23149.60	23265.90	23807.70	23910.50	
BW Interval	hr		2.50				3.00	3.75
Temp	C		23.00	25.00	24.00	23.00	30.90	
$CBOD_5$	Analyst		QW	CW	QW, SB	CW, SB, QW	QW, SB	
Sy In	mg/L		80.25	106.92	115.33	68.50	106.25	
Ry In	mg/L		< 3	5.66	3.09	3.22	7.56	
Ry Out	mg/L		< 3	3.71	3.21	< 3	4.79	
Sy Out	mg/L		3.16	3.75	< 3	3.15	4.25	
DO	Analyst		QW	CW	QW	SB	QW	QW
DO In	mg/L		8.97	4.53	5.57	5.38	5.58	*3.56*
DO Out	mg/L		7.20	1.20	3.38	2.72	2.53	*0.75*
OUR	$kg/m^3.d$		1.83	2.89	1.76	2.23	2.86	*2.21*
BODr	$kg/m^3.d$			1.69			2.60	
MX				0.58			0.91	
BOD Bed	mg/L			4.68	3.15		6.18	
Bed Load	$kg/m^3.d$			4.90	2.50	2.70	7.09	
TSS	Analyst		CW, MB		CW, MB		MB	
Sy In	mg/L		20.67		74.00		48.00	
Ry In	mg/L		1.56		4.00		4.89	
Ry Out	mg/L		1.89		3.00		2.67	
Sy Out	mg/L		0.78		2.56		3.22	
VSS	Analyst		CW, MB		CW, MB		MB	
Sy In	mg/L		20.00		68.00		50.00	
Ry In	mg/L		1.33		3.44		5.44	
Ry Out	mg/L		2.44		3.11		2.44	
Sy Out	mg/L		2.00		2.33		3.67	
Head Loss								
Top	cm							
Bed	cm							
Bottom	cm		0.50	1.0	1.0	1.0	0.5	1.0
Total	cm		12.00	16.0	15.5	15.5	16.0	18.0
pH								
Sy In								
Ry In								
Ry Out								
Sy Out								

BF4 October 2001 Data

Date	day		24-Oct-2001	25-Oct-2001	29-Oct-2001	31-Oct-2001	31-Oct-2001
	time		3:30 PM	2:00 PM	10:30 AM	11:30 AM	12:45 PM
Sampler			CW	SB, MB	QW	CW	CW
Qr	m^3/d		16.17	22.50	39.55	38.03	35.58
Sys Meter	gal		28123.80		28448.20	28809.10	28834.90
BW Interval	hr			2.00	1.75		2.00
Temp	C				15.35	18.00	19.00
CBOD$_5$	Analyst					CW,SB,QW	CW,SB,MB
Sy In	mg/L				97.67	~~107.48~~	~~104.54~~
Ry In	mg/L				7.18	~~12.28~~	~~10.28~~
Ry Out	mg/L				na	~~9.43~~	~~8.66~~
Sy Out	mg/L				7.07	~~8.21~~	~~8.89~~
DO	Analyst				QW	CW	CW
DO In	mg/L				6.82	*3.87*	3.95
DO Out	mg/L				5.73	*1.37*	1.47
OUR	kg/m^3.d				1.01	*2.24*	2.08
BODr	kg/m^3.d						
MX							
BOD Bed	mg/L						
Bed Load	kg/m^3.d						
TSS	Analyst						
Sy In	mg/L						
Ry In	mg/L						
Ry Out	mg/L						
Sy Out	mg/L						
VSS	Analyst						
Sy In	mg/L						
Ry In	mg/L						
Ry Out	mg/L						
Sy Out	mg/L						
Head Loss							
Top	cm					9.00	11.00
Bed	cm					11.00	9.00
Bottom	cm		1.00	0.5		1.0	1.0
Total	cm		18.00	8.0	16.0	22.0	21.0
pH							
Sy In							
Ry In							
Ry Out							
Sy Out							

BF4 November 2001 Data

Date	day / time		1-Nov-2001 11:00 AM	2-Nov-2001 11:00 AM	5-Nov-2001 2:30 PM	6-Nov-2001 11:00 AM	7-Nov-2001 4:00 PM	9-Nov-2001 12:30 PM
Sampler			SB	QW	CW, SB	SB, QW	CW	SB
Flow	m^3/d		14.01	15.16	22.25	23.79	17.28	12.92
Sys Meter	gal		29169.70	2942.20	29739.90	29864.90	30162.85	30563.80
BW Interval	hr			2.25		1.50	1.75	1.50
Temp	C		21.5	21.0	23.0		22.5	21.0
$CBOD_5$	Analyst					QW, CW	CW	CW, SB
Sy In	mg/L					95.50	100.56	114.68
Ry In	mg/L					8.01	15.34	15.24
Ry Out	mg/L					7.74	12.61	19.11
Sy Out	mg/L					6.21	10.23	
DO	Analyst			CW		QW	CW	SB
DO In	mg/L			2.87		4.42	2.57	1.98
DO Out	mg/L			1.63		1.10	0.00	0.00
OUR	$kg/m^3.d$			0.17		1.86	1.04	0.60
BODr	$kg/m^3.d$					0.15	1.11	-1.18
MX						0.08	1.06	-1.95
BOD Bed	mg/L					7.88	13.97	17.17
Bed Load	$kg/m^3.d$					4.49	6.24	4.63
TSS	Analyst			MB		MB	MB	MB
Sy In	mg/L			40.67		32.67	68.67	29.33
Ry In	mg/L			7.67		6.33	12.72	10.33
Ry Out	mg/L			5.44		7.78	8.27	13.44
Sy Out	mg/L			6.44		5.89	7.20	8.78
VSS	Analyst			MB		MB		
Sy In	mg/L			36.33		30.00		
Ry In	mg/L			9.00		5.78		
Ry Out	mg/L			6.22		7.67		
Sy Out	mg/L			5.67		5.56		
Head Loss								
Top	cm		12.00		13.00	12.00	20.00	12.00
Bed	cm		21.00		15.00	13.00	11.00	15.00
Bottom	cm		1.00		1.0	1.0	0.0	1.0
Total	cm		34.00		29.0	24.0	30.0	28.0
pH								
Sy In								
Ry In								
Ry Out								
Sy Out								

BF4 November 2001 Data

Date	day		13-Nov-2001	14-Nov-2001	14-Nov-2001	14-Nov-2001	14-Nov-2001
	time		11:40 AM	10:30 AM	12:30 PM	2:30 PM	4:30 PM
Sampler			SB	CW	CW	SB	SB
Flow	m^3/d		15.20	15.70	13.29	16.11	16.53
Sys Meter	gal		31563.00	31839.10	31851.70	31864.80	31903.00
BW Interval	hr		1.00	1.50	1.50	1.00	1.00
Temp	C		20.5	20.0	21.0	21.1	22.2
$CBOD_5$	Analyst			QW, SB	QW,CW	QW,SB	QW,CW
Sy In	mg/L			105.21	114.74	119.48	124.04
Ry In	mg/L			22.90	19.23	15<x<17.8	27.48
Ry Out	mg/L			20.35	20.35	22.73	28.78
Sy Out	mg/L			17.93			
DO	Analyst		QW	SB,CW	SB,CW	SB,CW	SB,CW
DO In	mg/L		5.17	1.45	1.87	2.32	1.82
DO Out	mg/L		2.10	0.00	0.10	0.05	0.00
OUR	$kg/m^3.d$		1.10	0.54	0.55	0.86	0.71
BODr	$kg/m^3.d$			0.94	-0.35		-0.51
MX				1.76	-0.63		-0.71
BOD Bed	mg/L			21.63	19.79		28.13
Bed Load	$kg/m^3.d$			8.46	6.01		10.69
TSS	Analyst			MB		MB	
Sy In	mg/L			38.00		54.00	
Ry In	mg/L			21.33		23.67	
Ry Out	mg/L			15.56		18.33	
Sy Out	mg/L			18.33			
VSS	Analyst			MB			
Sy In	mg/L			35.33		44.67	
Ry In	mg/L			19.67		15.33	
Ry Out	mg/L			11.33		14.33	
Sy Out	mg/L			15.33			
Head Loss							
Top	cm		7.00			12.00	14.00
Bed	cm		17.00			10.00	14.00
Bottom	cm		0.50			1.0	1.0
Total	cm		24.00			26.0	27.0
pH							
Sy In							
Ry In				7.55			
Ry Out				7.65			
Sy Out							

BF4 November 2001 Data

Date	day		16-Nov-2001	21-Nov-2001	21-Nov-2001	21-Nov-2001	21-Nov-2001	30-Nov-2001
	time		2:00 PM	10:45pm	12:45 PM	2:45 PM	4:45 PM	4:30 PM
Sampler			CW, SB	CW	CW	SB	SB	CW, SB
Flow	m^3/d			19.62	22.75	20.13	19.77	59.76
Sys Meter	gal		32377.45	33150.20	33160.85	33205.60	33231.10	35436.50
BW Interval	hr		1.00		1.00	1.25	1.00	1.00
Temp	C		22.0	15.5	16.8	18.2	18.8	18.2
$CBOD_5$	Analyst			CW,SB	CW,SB	CW, SB	CW,SB	CW,SB
Sy In	mg/L			100.85	76.10	71.30	91.20	87.90
Ry In	mg/L			9.68	13.79	9.37	12.33	14.09
Ry Out	mg/L			11.64	11.32	12.49	10.87	12.69
Sy Out	mg/L			9.33				13.21
DO	Analyst			SB,CW	SB,CW	SB,CW	SB,CW	SB
DO In	mg/L			5.28	4.37	4.37	4.28	4.30
DO Out	mg/L			2.05	1.27	1.20	1.02	2.92
OUR	$kg/m^3.d$			1.49	1.66	1.50	1.52	1.95
BODr	$kg/m^3.d$			-0.90	1.32	-1.48	0.68	1.97
MX				-0.61	0.80	-0.99	0.45	1.01
BOD Bed	mg/L			10.66	12.55	10.93	11.60	13.39
Bed Load	$kg/m^3.d$			4.47	7.38	4.44	5.74	19.82
TSS	Analyst			MB		MB	MB	
Sy In	mg/L			76.00		38.00	55.11	
Ry In	mg/L			12.00		9.78	10.00	
Ry Out	mg/L			13.67		10.00	12.33	
Sy Out	mg/L			11.00				
VSS	Analyst			MB		MB	MB	
Sy In	mg/L			62.67		42.00	48.89	
Ry In	mg/L			8.67		9.78	9.67	
Ry Out	mg/L			12.67		7.56	10.67	
Sy Out	mg/L			9.67				
Head Loss								
Top	cm						14.00	4.00
Bed	cm						14.00	4.00
Bottom	cm						1.0	1.0
Total	cm						24.0	8.0
pH								
Sy In								
Ry In								
Ry Out								
Sy Out								

BF4 December 2001 Data

Date	day		7-Dec-2001	11-Dec-2001	17-Dec-2001	18-Dec-2001	26-Dec-2001	27-Dec-2001
	time		1:00 PM	3:00 PM	12:45 PM	3:00 PM	3:15 PM	12:05 PM
Sampler			SB,MB	CW	CW,SB	CW,SB	SB	SB
Flow	m^3/d		54.16	60.19	52.43	47.60	36.34	38.02
Sys Meter	gal		36687.50	37361.25	38394.40	38713.60	39964.00	40118.70
BW Interval	hr		1.25	1.50	1.00	1.25		0.75
Temp	C		21.0	16.0	21.5	20.0	10.8	10.0
$CBOD_5$	Analyst		CW,SB	CW,SB	CW	CW,SB		SB
Sy In	mg/L		156.70	138.20	112.30	93.00		63.60
Ry In	mg/L		12.52	19.35	10.75	16.36		16.97
Ry Out	mg/L		12.07	18.33	9.47	13.86		15.18
Sy Out	mg/L		12.08	18.10	9.64	13.40		16.74
DO	Analyst		SB	SB	CW	SB		SB
DO In	mg/L		4.32	5.12	4.73	4.53		8.42
DO Out	mg/L		2.82	3.85	2.85	2.45		7.33
OUR	$kg/m^3.d$		1.91	1.79	0.87	2.33		0.97
BODr	$kg/m^3.d$		0.57	1.44	0.59	2.80		1.60
MX			0.30	0.80	0.68	1.20		1.65
BOD Bed	mg/L		12.30	18.84	10.11	15.11		16.08
Bed Load	$kg/m^3.d$		15.96	27.40	4.97	18.33		15.18
TSS	Analyst		MB	MB	MB	MB		MB
Sy In	mg/L		51.11	88.67	68.00	70.00		38.78
Ry In	mg/L		14.00	21.56	15.33	19.33		13.33
Ry Out	mg/L		11.56	19.33	9.67	15.00		9.83
Sy Out	mg/L		13.11	19.67	10.67	14.67		11.67
VSS	Analyst			MB	MB	MB		MB
Sy In	mg/L			84.00	62.00	64.67		37.89
Ry In	mg/L			21.33	14.33	20.33		15.50
Ry Out	mg/L			19.00	9.33	15.33		12.33
Sy Out	mg/L			19.00	10.67	15.00		13.50
Head Loss								
Top	cm		4.00	4.00	8.50	8.50		12.00
Bed	cm		4.00	5.00	7.50	8.00		6.00
Bottom	cm		1.00	1.0	1.0	1.0		1.0
Total	cm		8.00	10.0	14.5	17.0		22.0
pH								
Sy In								
Ry In								
Ry Out								
Sy Out								

BF4 January 2002 Data

Date	day		3-Jan-2002	4-Jan-2002	9-Jan-2002	10-Jan-2002	11-Jan-2002	15-Jan-2002
	time		2:00 PM	4:00 PM	3:00 PM	2:30 PM	3:00 PM	12:00 PM
Sampler			CW,SB	CW	CW,SB	CW,SB	CW,SB	CW,SB
Flow	m^3/d			45.93	51.56	37.32	30.84	64.45
Sys Meter	gal		41262.70	41367.70	42309.90	42520.00	42741.80	43444.30
BW Interval	hr		1.00	1.00	1.00	1.00	1.00	0.50
Temp	C			7.0	12.5	14.2	16.5	12.0
$CBOD_5$	Analyst			SB	SB,CW	SB,CW	SB,CW	
Sy In	mg/L			139.90	115.70	145.10	132.20	
Ry In	mg/L			32.05	22.26	17.56	13.00	
Ry Out	mg/L			31.65	20.89	16.53	12.36	
Sy Out	mg/L			30.83	22.80	16.38	< 12	
DO	Analyst			CW	CW	CW		
DO In	mg/L			8.82	6.52	5.33	5.23	
DO Out	mg/L			7.72	5.03	3.88	3.33	
OUR	$kg/m^3.d$			1.19	1.80	1.27	1.38	
BODr	$kg/m^3.d$			0.43	1.66	0.90	0.46	
MX				0.36	0.92	0.71	0.34	
BOD Bed	mg/L			31.85	21.58	17.05	12.68	
Bed Load	$kg/m^3.d$			34.64	27.01	15.42	9.44	
TSS	Analyst				CW	CW		
Sy In	mg/L				74.69	113.22		
Ry In	mg/L				19.39	20.00		
Ry Out	mg/L				18.33	17.14		
Sy Out	mg/L				19.26	18.00		
VSS	Analyst				CW	CW		
Sy In	mg/L				67.67	104.44		
Ry In	mg/L				17.44	19.45		
Ry Out	mg/L				17.00	15.56		
Sy Out	mg/L				17.78	16.80		
Head Loss								
Top	cm				8.50	11.50		
Bed	cm				5.50	5.50		
Bottom	cm				1.0	1.0		
Total	cm				15.5	16.5		
pH								
Sy In								
Ry In								
Ry Out								
Sy Out								

APPENDIX D: BF6 DATA

Legend for Use in Appendix D

YSI dissolved oxygen probes were occasionally used to determine the DO in the samples
This is indicated by use of italics.

italics = DO probe measurement

Analytical samples for DO, $CBOD_5$, TSS, and VSS were performed in triplicate, and in the case of $CBOD_5$, with two dilutions for all samples other than the system influent. Hach BOD QC samples were regularly analyzed for the $CBOD_5$ samples and standardization of the sodium thiosulfate titrant for Winkler's testing was consistently practiced. DO probes were regularly checked against Winkler titration. TSS and VSS samples were always accompanied with a blank, which was tested in triplicate along with the samples, as were the $CBOD_5$ samples.

The system was allowed to acclimate during November and the first part of December. During this time, DO concentrations inside the filter were occasionally taken along with BODs. While this data was not used for evaluation of the BF6 configuration, it is included for completeness.

~~number~~ = error with data

On a few occasions (such as December 27), the $CBOD_5$ samples were read at an incorrect time or the blank values were too high. The values obtained are shown, although they were not used in data analysis. Precise details explaining these data points are embedded as comments in the spreadsheet files on the companion CD for this document.

Comment on Sys Meter

The values given in this appendix are the total flow values recorded from the site visits. These numbers were processed to determine the daily flow by assuming that flow was generated for ten hours per day at the facility and that no flow was generated during weekends and over holidays when the facility was empty. These values may also be found on the companion CD.

BF6 November 2001 Data

Date	day		1-Nov-2001	2-Nov-2001	5-Nov-2001	13-Nov-2001	16-Nov-2001
	time		11:00 AM	11:00 AM	2:30 PM	11:40 AM	2:00 PM
Sampler			SB	QW	CW, SB	SB	CW, SB
Flow	m^3/d						
Sys Meter	gal						72444.20
BW Interval	hr						
Temp	C		21.5		23.0		
$CBOD_5$	Analyst		SB	CW, SB			
Sy In	mg/L		90.33				
Ry In	mg/L		4.18	5.88			
Ry Out	mg/L		5.06	6.11			
Sy Out	mg/L						
DO	Analyst		SB	QW		SB	
DO In	mg/L		*6.00*	*5.54*		*4.09*	
DO Out	mg/L		*5.07*	*5.18*		*1.36*	
OUR	$kg/m^3.d$						
BODr	$kg/m^3.d$						
MX							
BOD Bed	mg/L						
Bed Load	$kg/m^3.d$						
TSS	Analyst						
Sy In	mg/L						
Ry In	mg/L						
Ry Out	mg/L						
Sy Out	mg/L						
VSS	Analyst						
Sy In	mg/L						
Ry In	mg/L						
Ry Out	mg/L						
Sy Out	mg/L						
Head Loss							
Top	cm						
Bed	cm		3.5				
Bottom	cm						
Total	cm						
pH							
Sy In							
Ry In							
Ry Out							
Sy Out							

BF6 December 2001 Data

Date	day		7-Dec-2001	11-Dec-2001	17-Dec-2001	18-Dec-2001	26-Dec-2001	27-Dec-2001
	time		1:00 PM	3:00 PM	1:15 PM	3:00 PM	3:15 PM	12:00 PM
Sampler			SB,MB	CW	CW, SB	CW,SB	SB	SB
Qr	m^3/d				140.07	128.28	136.45	173.42
Sys Meter	gal			79551.4	82262.10	82813.70	84653.60	85414.80
BW Interval	hr				1.67	na	2.90	2.22
Temp	C				21.8	20.0	12.5	14.0
$CBOD_5$	Analyst				CW	CW, SB		SB
Sy In	mg/L				112.30	93.00		~~63.60~~
Ry In	mg/L				20.04	14.56		~~21.93~~
Ry Out	mg/L				17.96	16.43		~~19.16~~
Sy Out	mg/L				21.62	23.13		~~16.77~~
DO	Analyst		SB		CW	SB		SB
DO In	mg/L		3.49		1.25	2.07		3.10
DO Out	mg/L		1.95		0.20	1.02		2.02
OUR	$kg/m^3.d$				1.30	1.19		1.66
BODr	$kg/m^3.d$				2.56	-2.12		4.23
MX					1.97	-1.78		2.55
BOD Bed	mg/L				19.00	15.50		20.54
Bed Load	$kg/m^3.d$				24.77	16.48		33.56
TSS	Analyst				MB	MB		MB
Sy In	mg/L				68.00	70.00		~~38.78~~
Ry In	mg/L				11.17	40.00		~~18.39~~
Ry Out	mg/L				9.17	47.33		~~14.67~~
Sy Out	mg/L				15.33	62.67		~~10.44~~
VSS	Analyst				MB	MB		MB
Sy In	mg/L				62.00	64.67		~~37.89~~
Ry In	mg/L				11.17	31.67		~~20.28~~
Ry Out	mg/L				8.00	40.67		~~17.33~~
Sy Out	mg/L				15.67	55.56		~~14.89~~
Head Loss								
Top	cm							
Bed	cm							
Bottom	cm							
Total	cm							
pH								
Sy In								
Ry In								
Ry Out								
Sy Out								

BF6 January 2002 Data

Date	day		3-Jan-2002	4-Jan-2002	9-Jan-2002	10-Jan-2002	11-Jan-2002	14-Jan-2002
	time		2:00 PM	3:15 PM	3:00 PM	3:00 PM	3:00 PM	3:30 PM
Sampler			SB, CW	CW	CW,SB	SB,CW	CW,SB	SB
Flow	m^3/d			118.81	118.05	107.54	125.36	
Sys Meter	gal		87517.80	88284.80	90833.80	91611.50	92244.50	93630.60
BW Interval	hr		2.22	2.96	2.22			
Temp	C		9.0	10.8	14.0	16.0	17.5	
$CBOD_5$	Analyst			SB,CE	SB,CW	SB,CW	CW,SB	
Sy In	mg/L			139.90	115.70	145.10	132.20	
Ry In	mg/L			45.38	17.71	> 59	> 42	
Ry Out	mg/L			39.95	14.24	> 56	> 43	
Sy Out	mg/L			40.73	14.46	> 56	> 43	
DO	Analyst			CW	CW	CW	SB	
DO In	mg/L			3.42	2.67	0.62	0.50	
DO Out	mg/L			2.73	1.62	0.07	0.07	
OUR	$kg/m^3.d$			0.72	1.09	0.52	0.48	
BODr	$kg/m^3.d$			5.69	3.61			
MX				7.94	3.30			
BOD Bed	mg/L			42.66	15.97			
Bed Load	$kg/m^3.d$			47.58	18.45			
TSS	Analyst				CW	CW		
Sy In	mg/L				74.69	113.22		
Ry In	mg/L				16.93	50.33		
Ry Out	mg/L				13.78	41.33		
Sy Out	mg/L				12.38	52.62		
VSS	Analyst				CW	CW		
Sy In	mg/L				67.67	104.44		
Ry In	mg/L				15.07	49.33		
Ry Out	mg/L				12.56	38.44		
Sy Out	mg/L				11.71	51.48		
Head Loss								
Top	cm							
Bed	cm				6.50			
Bottom	cm							
Total	cm							
pH								
Sy In								
Ry In								
Ry Out								
Sy Out								
Backmix	m^3/d							

BF6 January 2002 Data

Date	day		15-Jan-2002	17-Jan-2002	18-Jan-2002	24-Jan-2002	25-Jan-2002	31-Jan-2002
	time		12:00 PM	5:00 PM	2:30 PM	4:15 PM	11:30 AM	3:00 PM
Sampler			CW,SB	SB,CW	CW	CW	SB	CW
Flow	m^3/d		122.96		83.13	93.99	92.48	105.97
Sys Meter	gal		94064.10	95498.90	95678.60	97067.90	97165.40	97900.40
BW Interval	hr				too low	1.91	1.91	2.67
Temp	C		14.0	17.0	17.0	22.5	16.0	23.2
$CBOD_5$	Analyst				CW,SB	CW,SB	CW, SB	CW,SB
Sy In	mg/L				152.50	164.40	148.82	141.40
Ry In	mg/L				< 6	6.54	13.67	< 3
Ry Out	mg/L				< 6	5.33	11.22	< 3
Sy Out	mg/L				< 6	6.29	11.23	< 3
DO	Analyst		CW		CW	CW	SB	CW
DO In	mg/L		1.50		5.58	4.98	6.08	4.62
DO Out	mg/L		0.62		3.13	2.88	5.10	3.02
OUR	$kg/m^3.d$		0.96		1.80	1.73	0.80	1.50
BODr	$kg/m^3.d$					1.01	2.00	
MX						0.58	2.49	
BOD Bed	mg/L					5.93	12.45	
Bed Load	$kg/m^3.d$					5.42	11.16	
TSS	Analyst					CW	CW	CW
Sy In	mg/L					81.03	102.55	80.56
Ry In	mg/L					14.52	34.40	2.18
Ry Out	mg/L					13.07	32.89	1.87
Sy Out	mg/L					15.76	27.17	2.00
VSS	Analyst					CW	CW	CW
Sy In	mg/L					74.36	94.12	78.89
Ry In	mg/L					12.74	31.20	2.18
Ry Out	mg/L					11.33	29.00	1.98
Sy Out	mg/L					12.82	24.75	2.00
Head Loss								
Top	cm							
Bed	cm							
Bottom	cm							
Total	cm							
pH								
Sy In								
Ry In								
Ry Out								
Sy Out								
Backmix	m^3/d				4.98	3.72		6.01

BF6 February 2002 Data

Date	day		1-Feb-2002	6-Feb-2002	7-Feb-2002	8-Feb-2002	13-Feb-2002	14-Feb-2002
	time		10:30 AM	4:00 PM	4:00 PM	1:00 PM	5:00 PM	9:30 AM
Sampler			SB	MC	CW,SB	SB	MC	SB
Flow	m^3/d		94.78		100.42	80.77		84.30
Sys Meter	gal		98058.20	98862.00	99150.00	99564.10	101900.00	102142.60
BW Interval	hr		2.67		2.67	2.67		3.33
Temp	C		16.5		10.8	12.5		14.0
$CBOD_5$	Analyst		SB, CW		SB,CW	SB,SA		CW
Sy In	mg/L		117.50		156.60	132.50		144.30
Ry In	mg/L		6.13		18.91	> 20		> 22
Ry Out	mg/L		3.61		15.49	> 20		18.57
Sy Out	mg/L		< 3		15.87	> 20		15.23
DO	Analyst		CW		CW	CW		CW
DO In	mg/L		5.75		6.25	3.98		5.05
DO Out	mg/L		4.05		5.03	2.75		2.17
OUR	$kg/m^3.d$		1.42		1.08	0.88		2.15
BODr	$kg/m^3.d$		2.11		3.03			
MX			1.48		2.81			
BOD Bed	mg/L		4.87		17.20			18.57
Bed Load	$kg/m^3.d$		5.13		16.76			
TSS	Analyst		CW		CW, SA	CW		CW
Sy In	mg/L		43.74		233.33	89.34		86.17
Ry In	mg/L		13.78		39.55	47.40		36.00
Ry Out	mg/L		8.57		33.11	41.96		28.67
Sy Out	mg/L		2.41		24.13	37.56		23.58
VSS	Analyst		CW		CW, SA	CW		CW
Sy In	mg/L		37.67		195.56	79.20		76.08
Ry In	mg/L		12.56		34.01	41.20		31.11
Ry Out	mg/L		8.14		28.44	36.87		23.83
Sy Out	mg/L		2.73		21.90	31.56		19.18
Head Loss								
Total	cm							15.5
pH								
Ry In					7.17			
Ry Out					7.13			
Backmix	m^3/d		1.74		4.30	2.72		3.46
Filtered $CBOD_5$					SB,CW			
Sy In	mg/L				78.33			
Ry In	mg/L				4.15			
Ry Out	mg/L				3.3			
Sy Out	mg/L				4.5			

BF6 February 2002 Data

Date	day time		15-Feb-2002 3:30pm	20-Feb-2002 3:30 PM	21-Feb-2002 4:20 PM	22-Feb-2002 3:30 PM	28-Feb-2002 3:10 PM
Sampler			CW	CW	SB	CW,SA	SB
Flow	m^3/d		83.27		59.08	58.08	49.90
Sys Meter	gal		103300.80	107666.50	107988.60	108443.00	110458.80
BW Interval	hr		3.81	2.43	4.44	5.12	7.38
Temp	C		17.5	21.5	20.5	17.8	15.0
$CBOD_5$	Analyst		CW,SB		SB,SA,CW	CW	CW,SA
Sy In	mg/L		176.00		148.80	151.40	168.20
Ry In	mg/L		> 50		20.87	43.55	53.86
Ry Out	mg/L		> 50		16.67	36.20	45.63
Sy Out	mg/L		> 50		14.48	40.65	31.86
DO	Analyst		CW		SB	CW	SB
DO In	mg/L		0.28		3.45	3.12	2.88
DO Out	mg/L		0.00		1.13	0.88	0.60
OUR	$kg/m^3.d$		0.21		1.21	1.14	1.01
BODr	$kg/m^3.d$				2.19	3.77	3.62
MX					1.82	3.29	3.60
BOD Bed	mg/L				18.77	39.88	49.74
Bed Load	$kg/m^3.d$				10.88	22.32	23.72
TSS	Analyst		CW		CW	CW	SA
Sy In	mg/L		124.00		73.00	79.94	83.33
Ry In	mg/L		110.00		39.39	31.02	54.00
Ry Out	mg/L		98.89		37.07	23.50	40.95
Sy Out	mg/L		102.62		35.90	25.86	18.86
VSS	Analyst		CW		CW	CW	SA
Sy In	mg/L		116.00		64.78	76.24	75.56
Ry In	mg/L		100.83		34.72	28.34	48.67
Ry Out	mg/L		90.56		32.53	21.62	37.72
Sy Out	mg/L		94.68		30.26	24.05	16.76
Head Loss							
Top	cm						
Bed	cm						
Bottom	cm						
Total	cm				20.0		
pH							
Sy In							
Ry In			7.35		7.62	7.08	
Ry Out			7.32		7.55	7.06	
Sy Out			7.38			7.12	
Backmix	m^3/d				0.81	1.59	2.00

BF6 March 2002 Data

Date	day		1-Mar-2002	4-Mar-2002	7-Mar-2002	12-Mar-2002	14-Mar-2002	15-Mar-2002
	time		3:00 PM	1:15 PM	2:50 PM	1:30 PM	1:30 PM	1:10 PM
Sampler			CW	SB	SB	CW	CW	SB
Flow	m^3/d		42.72		61.11		60.31	
Sys Meter	gal		111038.5	112229.2	114183.8	116185.7	117550.0	18347.0
BW Interval	hr		13.25	1.91	3.33	2.22	2.22	2.90
Temp	C		13.8		19.5		19.2	
$CBOD_5$	Analyst		CW,SB		CW,SB		CW,SB	
Sy In	mg/L		198.00		197.20		141.38	
Ry In	mg/L		> 55		82.48		94.73	
Ry Out	mg/L		> 55		> 80		81.93	
Sy Out	mg/L		> 55		68.24		67.47	
DO	Analyst		SB				SB	
DO In	mg/L		2.43		0.40		0.00	
DO Out	mg/L		0.30		0.00		0.00	
OUR	$kg/m^3.d$		0.80		0.22		0.00	
BODr	$kg/m^3.d$						6.81	
MX								
BOD Bed	mg/L						88.33	
Bed Load	$kg/m^3.d$						50.42	
TSS	Analyst		CW,SA		SA		SA	
Sy In	mg/L		102.42		122.67		100.00	
Ry In	mg/L		65.33		65.33		58.67	
Ry Out	mg/L		52.22		70.67		49.78	
Sy Out	mg/L		61.11		52.67		60.00	
VSS	Analyst		CW,SA		SA		SA	
Sy In	mg/L		81.21		112.00		82.67	
Ry In	mg/L		57.33		60.89		49.78	
Ry Out	mg/L		45.56		66.67		39.56	
Sy Out	mg/L		54.67		51.33		49.33	
Head Loss								
Top	cm							
Bed	cm							
Bottom	cm							
Total	cm				20.0		23.0	
pH								
Sy In								
Ry In			7.39		7.61			
Ry Out			7.23		7.55			
Sy Out			7.26					
Backmix	m^3/d		1.32				2.06	

BF6 March 2002 Data

Date	day		21-Mar-2002	22-Mar-2002	27-Mar-2002	28-Mar-2002	28-Mar-2002
	time		12:00 PM	3:15 PM	3:40 PM	2:00 PM	2:00 PM
Sampler			CW	CW,SB	CW	CW	
Flow	m^3/d			60.93		55.23	
Sys Meter	gal		119870.0	120410.9	121522.6	121884.5	
BW Interval	hr		2.90	2.96	2.05	2.15	
Temp	C			20.0		20.5	
							FILTERED
$CBOD_5$	Analyst			CW,SB		SB,CW	SB,CW
Sy In	mg/L			183.80		142.20	114.83
Ry In	mg/L			> 70		21.27	5.78
Ry Out	mg/L			> 70		17.13	5.38
Sy Out	mg/L			> 70		11.68	< 5
DO	Analyst			CW		CW	
DO In	mg/L			1.18		3.68	
DO Out	mg/L			0.00		0.13	
OUR	$kg/m^3.d$			0.64		1.73	
BODr	$kg/m^3.d$					2.01	
MX						1.16	
BOD Bed	mg/L					19.20	
Bed Load	$kg/m^3.d$					10.37	
TSS	Analyst			CW,SA		CW,SA	
Sy In	mg/L			110.67		71.00	
Ry In	mg/L			48.00		39.72	
Ry Out	mg/L			38.89		30.44	
Sy Out	mg/L			44.22		16.79	
Sy Out + 30 mi	mg/L			21.45			
VSS	Analyst			CW,SA		CW,SA	
Sy In	mg/L			92.00		61.67	
Ry In	mg/L			44.00		37.22	
Ry Out	mg/L			36.67		28.67	
Sy Out	mg/L			41.56		16.45	
Head Loss							
Top	cm						
Bed	cm						
Bottom	cm			0.3			
Total	cm			24.0			
pH							
Sy In						6.51	
Ry In				7.10		7.14	
Ry Out				7.05		7.09	
Sy Out				7.27		7.29	
Backmix	m^3/d			2.89		1.61	

BF6 April 2002 Data

Date	day		1-Apr-2002	2-Apr-2002	3-Apr-2002	4-Apr-2002	4-Apr-2002	11-Apr-2002
	time		11:45 PM	2:15 PM	3:00 PM	11:45 AM	11:45 AM	4:05 PM
Sampler			CW	CW	SB	SB		SB
Flow	m^3/d			79.00		80.97		
Sys Meter	gal		122960.0	123574.8	126330.1	126533.5		130415.9
BW Interval	hr		1.96	2.38	2.15	1.49		1.52
Temp	C			22.8	24.8	17.5		24.0
							FILTERED	
$CBOD_5$	Analyst			CW,SB		SB,CW		
Sy In	mg/L			88.00		75.80	55.28	
Ry In	mg/L			15.07		13.38	< 4	
Ry Out	mg/L			12.41		9.24	< 4	
Sy Out	mg/L			9.24		10.16	< 4	
DO	Analyst			CW		SB		
DO In	mg/L			2.65		5.97		
DO Out	mg/L			0.33		4.52		
OUR	$kg/m^3.d$			1.62		1.04		
BODr	$kg/m^3.d$			1.85		2.96		
MX				1.15		2.86		
BOD Bed	mg/L			13.74		11.31		
Bed Load	$kg/m^3.d$			10.50		9.56		
TSS	Analyst			SA		SA		
Sy In	mg/L			51.19		38.52		
Ry In	mg/L			20.83		37.56		
Ry Out	mg/L			14.46		26.92		
Sy Out	mg/L			13.18		26.53		
VSS	Analyst			SA		SA		
Sy In	mg/L			42.96		32.96		
Ry In	mg/L			19.33		37.78		
Ry Out	mg/L			12.13		23.08		
Sy Out	mg/L			11.21		22.13		
Head Loss								
Top	cm							
Bed	cm							
Bottom	cm							
Total	cm							
pH								
Sy In								
Ry In								
Ry Out								
Sy Out								
Backmix	m^3/d			1.59		2.54		

BF6 April 2002 Data

Date	day		12-Apr-2002	15-Apr-2002	18-Apr-2002	18-Apr-2002	19-Apr-2002	3-May-2002
	time		5:30 PM	3:15 PM	4:00 PM	4:00 PM	3:00 PM	taken off line
Sampler			CW	SB,CW	CW		SB	
Flow	m^3/d		155.78		122.14			
Sys Meter	gal		130964.8	131573.1	133474.5		140592.4	
BW Interval	hr		1.31	1.49	1.41		2.22	
Temp	C		23.8		27.0			
						FILTERED		
$CBOD_5$	Analyst		CW,SB,SA		CW,SB	CW,SB		
Sy In	mg/L		121.90		141.00	92.89		
Ry In	mg/L		36.57		19.55	6.14		
Ry Out	mg/L		31.57		16.80	5.82		
Sy Out	mg/L		30.80		17.41	6.19		
DO	Analyst		CW		CW			
DO In	mg/L		3.05		2.98			
DO Out	mg/L		3.02		2.50			
OUR	$kg/m^3.d$		0.04		0.52			
BODr	$kg/m^3.d$		6.87		2.96			
MX			153.06		5.68			
BOD Bed	mg/L		34.07		18.17			
Bed Load	$kg/m^3.d$		50.28		21.07			
TSS	Analyst		SA		SA			
Sy In	mg/L		51.42		74.87			
Ry In	mg/L		31.00		29.33			
Ry Out	mg/L		26.17		22.96			
Sy Out	mg/L		29.81		26.35			
VSS	Analyst		SA		SA			
Sy In	mg/L		50.75		68.72			
Ry In	mg/L		28.52		27.11			
Ry Out	mg/L		23.83		21.11			
Sy Out	mg/L		27.82		24.13			
Head Loss								
Top	cm							
Bed	cm							
Bottom	cm							
Total	cm							
pH								
Sy In								
Ry In								
Ry Out								
Sy Out								
Backmix	m^3/d		3.41		4.18			

APPENDIX E: STATISTICAL OUTPUT FROM SPSS

Regression: Reactor BF4 with Temperature < 21C

Variables Entered/Removed[b]

Model	Variables Entered	Variables Removed	Method
1	LOAD_L21[a]	.	Enter

a. All requested variables entered.

b. Dependent Variable: BODR_L21

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.105[a]	.011	-.130	.84904

a. Predictors: (Constant), LOAD_L21

ANOVA[b]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.056	1	.056	.077	.789[a]
	Residual	5.046	7	.721		
	Total	5.102	8			

a. Predictors: (Constant), LOAD_L21

b. Dependent Variable: BODR_L21

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	1.052	.735		1.431	.196
	LOAD_L21	9.633E-03	.035	.105	.278	.789

a. Dependent Variable: BODR_L21

Regression: Reactor BF4 with Temperature > 21C

Variables Entered/Removed[b]

Model	Variables Entered	Variables Removed	Method
1	LOAD_G21[a]	.	Enter

a. All requested variables entered.

b. Dependent Variable: BODR_G21

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.947[a]	.897	.888	.88514

a. Predictors: (Constant), LOAD_G21

ANOVA[b]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	81.789	1	81.789	104.392	.000[a]
	Residual	9.402	12	.783		
	Total	91.190	13			

a. Predictors: (Constant), LOAD_G21

b. Dependent Variable: BODR_G21

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-1.084	.409		-2.647	.021
	LOAD_G21	.380	.037	.947	10.217	.000

a. Dependent Variable: BODR_G21

Regression: Reactor BF6 with Temperature < 21C

Variables Entered/Removed[b]

Model	Variables Entered	Variables Removed	Method
1	LOAD_L21[a]	.	Enter

a. All requested variables entered.

b. Dependent Variable: BODR_L21

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.970[a]	.940	.933	.40386

a. Predictors: (Constant), LOAD_L21

ANOVA[b]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	22.970	1	22.970	140.831	.000[a]
	Residual	1.468	9	.163		
	Total	24.438	10			

a. Predictors: (Constant), LOAD_L21

b. Dependent Variable: BODR_L21

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	1.379	.212		6.512	.000
	LOAD_L21	9.996E-02	.008	.970	11.867	.000

a. Dependent Variable: BODR_L21

Regression: Reactor BF6 with Temperature > 21C

Variables Entered/Removed[b]

Model	Variables Entered	Variables Removed	Method
1	LOAD_G21[a]	.	Enter

a. All requested variables entered.

b. Dependent Variable: BODR_G21

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.919[a]	.845	.768	.41265

a. Predictors: (Constant), LOAD_G21

ANOVA[b]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	1.861	1	1.861	10.930	.081[a]
	Residual	.341	2	.170		
	Total	2.202	3			

a. Predictors: (Constant), LOAD_G21

b. Dependent Variable: BODR_G21

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	.745	.457		1.629	.245
	LOAD_G21	8.742E-02	.026	.919	3.306	.081

a. Dependent Variable: BODR_G21

VITA

Born on July 16, 1979, in Baton Rouge, Louisiana, Cynthia Ann Wagener is the daughter of Gerald and Beverly Wagener. She was raised in Baton Rouge and attended Baton Rouge Magnet High School, from which she graduated in 1997. During the summer of 1996, she attended Birmingham Southern College. Following high school, she enrolled at Louisiana State University and in December of 2000, she received a bachelor's degree in environmental engineering. Because of the encouragement she received from her undergraduate professors, Cynthia was inspired to attend graduate school at LSU. As a graduate student, she enjoyed great academic freedom and was able to spend the summer of 2002 in southern Germany and co-taught EVEG 3110 in the spring and fall semesters of 2003 with her mentor and major professor, Dr. Ronald Malone. Cynthia has been a full time graduate student at Louisiana State University since Spring 2001, and is presently a candidate for the degree of Master of Science in Civil Engineering.